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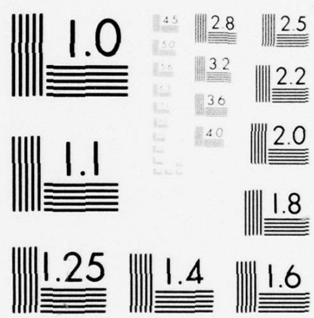

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AIRBLAST SIMULATOR DESIGN

Civil/Nuclear Systems Corporation
Albuquerque, NM 87102

July 1978

Final Report

Approved for public release; distribution unlimited.

This research was sponsored by the Defense Nuclear Agency under Subtask SX 355, Work Units 2, 10, 11 Work Unit Titles: Trench Concept Development, Shelter Concept Development, and 0704 Trench Concept Development.

Prepared for
Director
DEFENSE NUCLEAR AGENCY
Washington, DC 20305

AIR FORCE WEAPONS LABORATORY
Air Force Systems Command
Kirtland Air Force Base, NM 87117

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This technical report has been reviewed and is approved for publication.

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		High and Low Pressure
Clear-span Structures	Nuclear Simulation	Air Shock
Simulators	Arches	Cavity
HEST (High Explosive Simulation Technique)	Facilities	Overpressure
DABS (Dynamic Airblast Simulator)	Explosive Driver	Dynamic Pressure
Airblast	Overburden	Shock Tube

20. ABSTRACT (Continue on reverse side if necessary and identify by block number) (UNCLASSIFIED ABSTRACT)

This study was undertaken to identify structural and simulator concepts for use in the development of designs for low-cost disposable airblast simulator facilities. Specifically addressed were the Dynamic Airblast Simulator (DABS), an explosively-driven shock tube, and the High Explosive Simulation Technique (HEST) an earth-covered, explosive cavity. For the DABS, alternative construction concepts, including parabolic arches, cable suspension systems, concrete box girders, long-span steel joists, composite deck system, aircraft shelter arches and

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circular concrete arches were evaluated for construction and cost feasability. High pressure HEST facility concepts and the use of HEST for testing aboveground structures were evaluated. The various airblast simulator concepts were compared and rated relative to one another.

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ILLUSTRATIONS (concluded)

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SECTION I

INTRODUCTION

1. OBJECTIVE

The objective of this study was to identify structural and simulator concepts for use in the development of designs for low-cost disposable airblast simulator facilities. Such simulator facilities are used for testing large-scale protective structures in a realistic airblast environment.

The DABS is an earth covered explosive-driven shock tube with the DABS facility being the tube through which the air shock is driven. In the DABS, the quantity of explosives, the placement of the test item in the facility, the depth of earth overburden, and the length of the simulator are varied to obtain the desired airblast simulation.

The HEST is an earth covered cavity in which explosives are placed. The quantity of explosives, distribution of the explosives in the cavity, the cavity depth and the depth of overburden are varied to obtain the desired airblast characteristics.

At the time of this study, neither the DABS nor HEST techniques had been used at the pressure levels and sizes required to adequately test the basing concepts for the Multiple Aim Point Program (MX). The DABS concept has been used in tests of small-scale MX model structures. The HEST concept has been used for tests of surface flush or buried structures.

This study was divided into two general areas of investigation. The major effort was directed towards developing structural concepts for large-scale DABS facilities. A related effort was a theoretical evaluation of the HEST technique for simulation of dynamic airblast loading of aboveground structures. The other area of investigation studied structural concepts for a high pressure HEST facility for possible use in testing buried portions of the MX concept.

2. ORGANIZATION OF REPORT

Section II of this report presents the development and evaluation of high pressure HEST facility concepts. An investigation of the feasibility of using the HEST for dynamic airblast simulation in aboveground structure tests is contained in Section III. Section IV presents the development of seven DABS concepts, and Section V summarizes the evaluation of the more promising concepts. A summary of the results and conclusions of the study is presented in Section VI.

SECTION II

HIGH PRESSURE HEST FACILITY CONCEPTS

1. GENERAL

Structural concepts for a high pressure HEST facility to be used in the MX In-Trench Environment Definition Test are presented in this section. In order to develop the 500,000 psi to 500 psi pressure range required for this test, it is expected that slurry or solid explosive charges will be used in the higher pressures regions of the HEST facility. Detonating cord matrices would satisfy requirements in the lower pressure regions.

Four concepts were selected for evaluation as to feasibility for the test. The first three concepts are variations of proven designs used in earlier HEST facilities. The fourth concept is an egg-crate design which, although unproven, appears to be feasible. Each concept was developed within the following general guidelines and design criteria. The general guidelines were

- Conventional construction methods and materials shall be used.
- Existing column/girder/beam designs shall be considered.
- Concepts utilizing timber, steel, concrete and combinations thereof shall be evaluated.

- The basic facility design parameters and explosives requirements, including timing and firing systems, shall be provided by the Air Force.

The design criteria for the test facility were

- Length - 1000 ft.
- Width - 48 ft.
- Depth - Variable from 0.4 ft to 1.6 ft.
- Overburden - 1000 psf.
- Soil bearing capacity, q_{ult} = 4400 psf.

Because of the small cavity depth, small structural settlements or deflections could cause relatively large percentage changes in the cavity volume and charge density. Therefore, close attention must be given to these details in the final analysis of the test facility.

Selection of a recommended facility concept was based upon cost and performance. Conformance to facility design parameters and compatibility with the proposed distribution, timing and firing of the explosives were included in the basic performance criteria. Factors influencing construction costs are discussed later.

2. DEBRIS FALLBACK MINIMIZATION

Material fallback onto the test bed is a problem common to all HEST concepts. Since this fallback complicates reentry and can cause damage to test structures, some attention was given to methods for minimizing the amount of fallback

that does occur. Methods of minimizing the amount of fallback are influenced to some extent by the HEST cavity requirements. In most HEST tests, the ground surface within the test bed area must be reasonably level. The depth of soil cover is controlled by the required surcharge weight so that the soil cover is also fairly flat. The variable cavity height is normally obtained by sloping the cavity roof along the longitudinal axis of the array.

The most attractive and practical proposals for minimizing fallback rely upon interaction between the airblast and surcharge support structure. Since the support structure is generally composed of numerous individual uncoupled components, it is difficult to cause the general mass to move in any direction other than upward. If the surcharge support structure elements are interconnected to form a somewhat continuous structure, it might be possible to move a large portion of the surcharge laterally to the sides of the test bed. For example, if the support structure is broken at its centerline and temporarily restrained along its edges, it might be possible to impart a rotation to the soil cover which would carry it to the side of the test bed. One method of restraining the edges of the support structure system might involve the placement of woven reinforcing, such as welded wire fabric or cyclone fencing, into the soil cover as shown in figure 1. If the mesh is extended from the centerline to beyond the edge

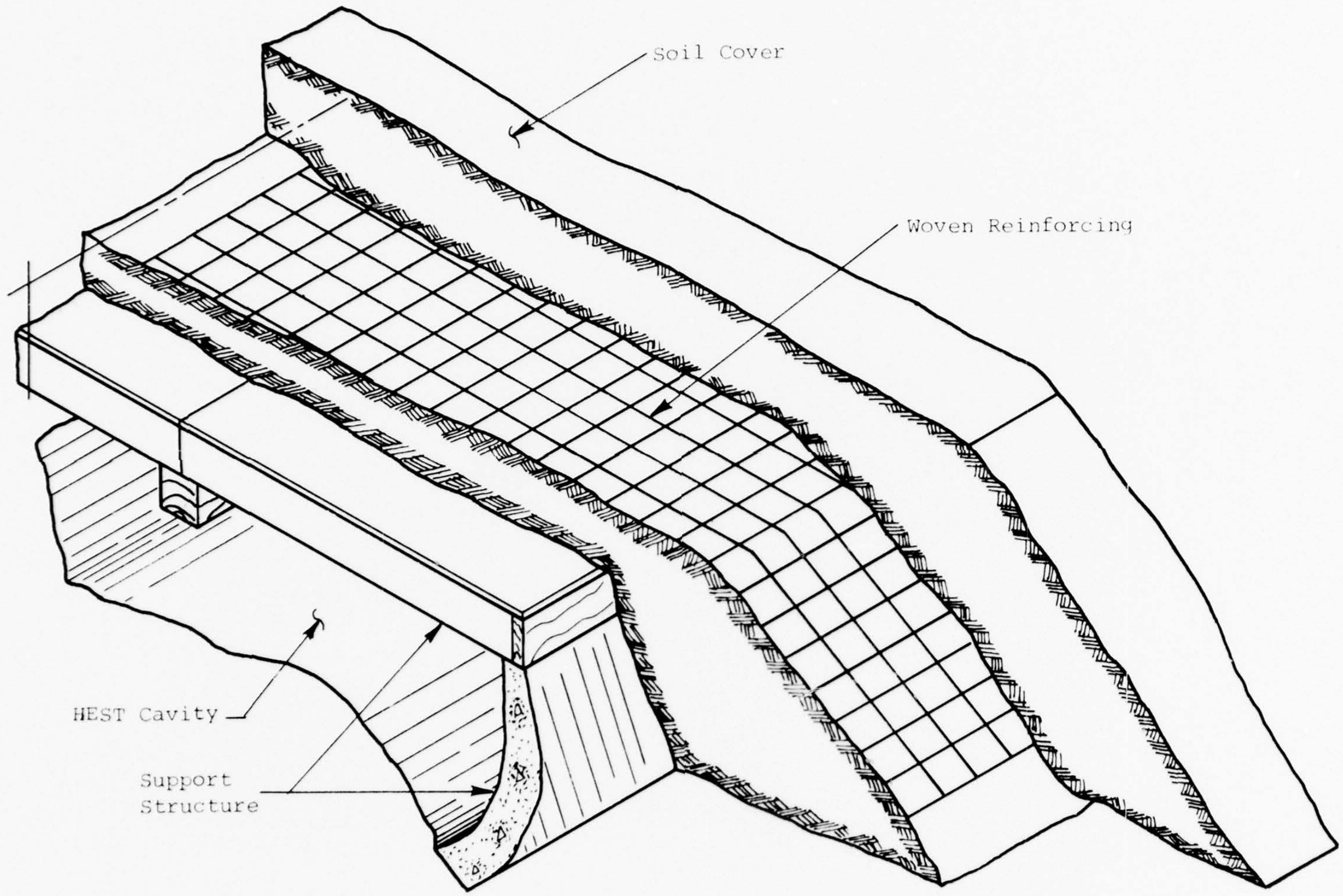
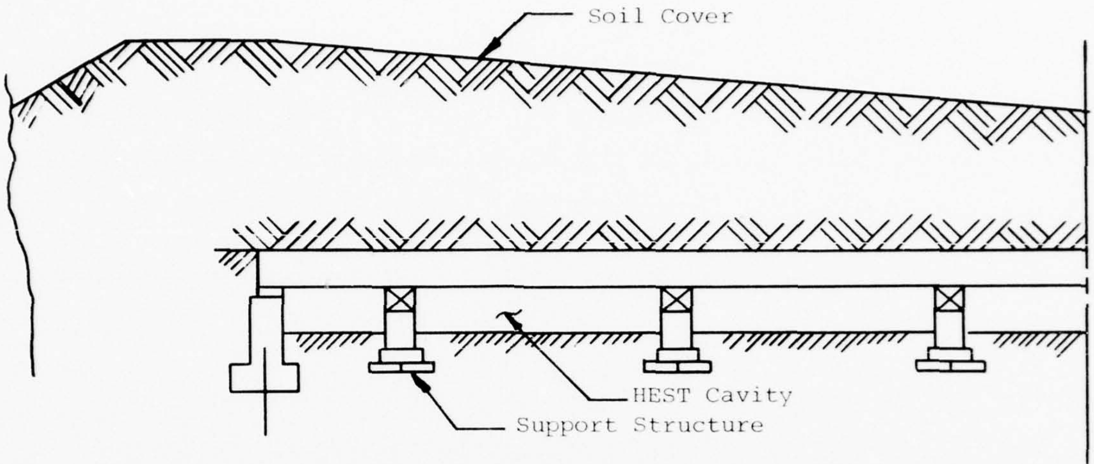


Figure 1. Woven Reinforcing Edge Restraint

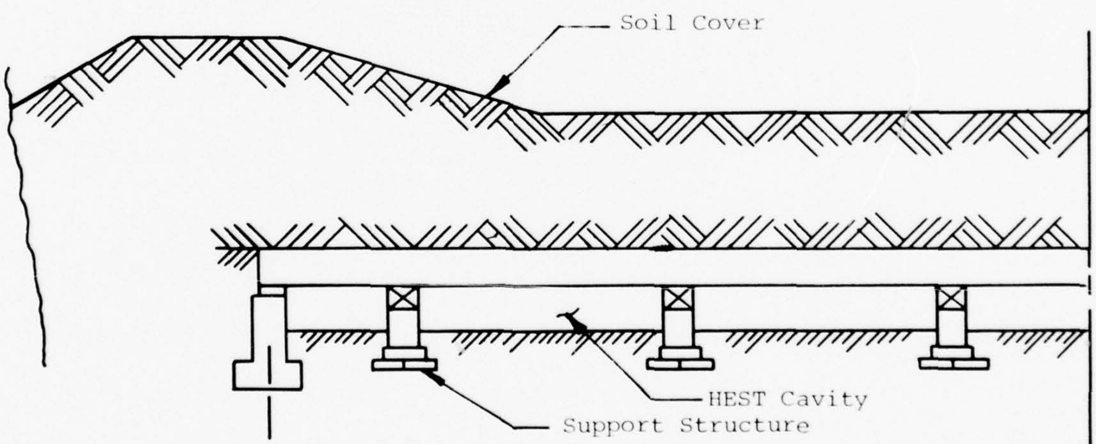
of each half of the test bed, sufficient momentary restraint might be developed to induce rotation.

Another approach would involve placing less soil cover along the centerline of the test bed than along the sides. Two variations of this approach, a linear taper and an edge concentration, are shown in figure 2. Using this approach, the difference in locations of the center of gravity of the surcharge mass and the resultant of blast pressures acting on the bottom of the surcharge support structure could be enough to initiate the small amount of rotation required. Providing some continuity in the surcharge or surcharge support structure would increase the effectiveness of this scheme. A variable depth surcharge may have contributed to the minimization of fallback in recent DABS tests at the University of New Mexico Civil Engineering Research Facility. The effectiveness of either of these methods would require experimental evaluation, since available analytical methods do not treat the problem satisfactorily.

Tilting the entire test bed at an angle, as shown in figure 3, would also tend to cause the soil cover to be ejected to one or both sides of the test structures. An angle of 10 degrees should be adequate to minimize fallback. Such a slope should not greatly affect construction costs and would still permit the use of conventional construction techniques. The measurements program would have to include consideration of



a. Linear Taper



b. Edge Concentration

Figure 2. Increased Soil Cover Edge Restraint

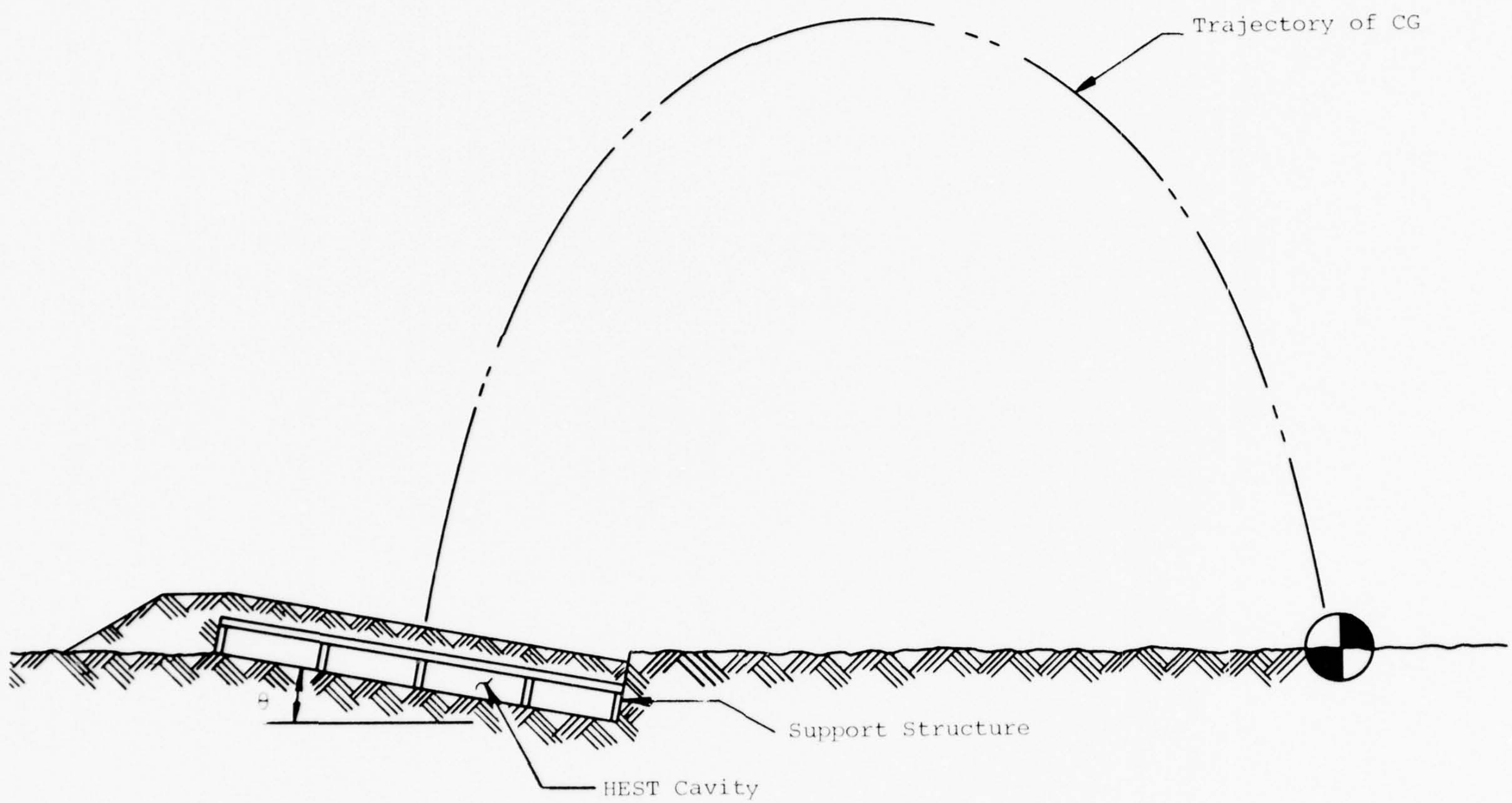


Figure 3. Sloped Test Bed to Reduce Fallback

the test bed slope. A sloped test bed may create problems in other areas of the program which would prevent the use of this method of minimizing fallback.

One other concept of surcharge dispersal is the use of polyethylene sheets embedded in the soil surcharge (fig. 4). The sloping polyethelene sheets are intended to provide a wedging action which moves portions of the overburden laterally towards the sides of the test bed. This concept was employed in several events of the HARD PAN I series, but there is some difference of opinion as to its effectiveness. Although there is some evidence of a reduction in the amount of debris which falls inside the test bed, a considerable amount remained in the HARD PAN I Event 3 test bed.

3. DESCRIPTION OF PROPOSED CONCEPTS

All structural elements have been designed to carry a surcharge of 1000 psf. A load factor of one was utilized for design purposes. Normal allowable design stresses for wood structures were multiplied by a factor of 1.15 in recognition of the short-term nature of the surcharge loads. The factor of safety against failure is somewhat greater than one because of the conservatism in specifying allowable stresses. All sawn timber is specified to be Douglas Fir, Coast Region, Dense Select Structural Grade. All plywood is specified to be Exterior Structural I Grade, Species Group 1.

All concepts have a reinforced concrete wall around the perimeter of the test bed.

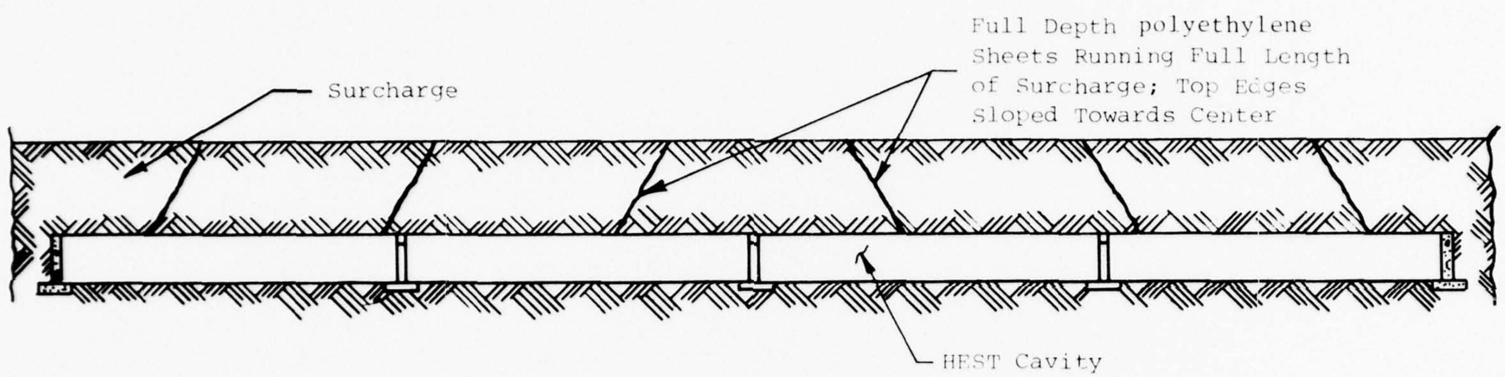


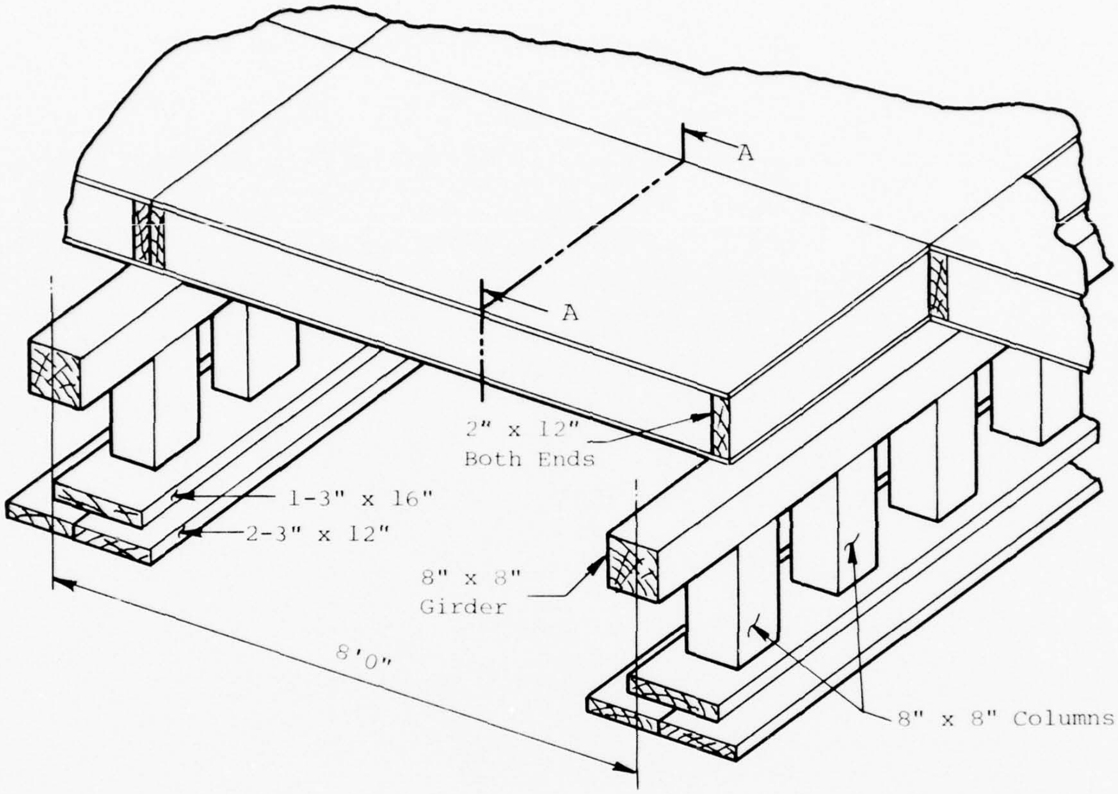
Figure 4. Surcharge Dispersal Membrane

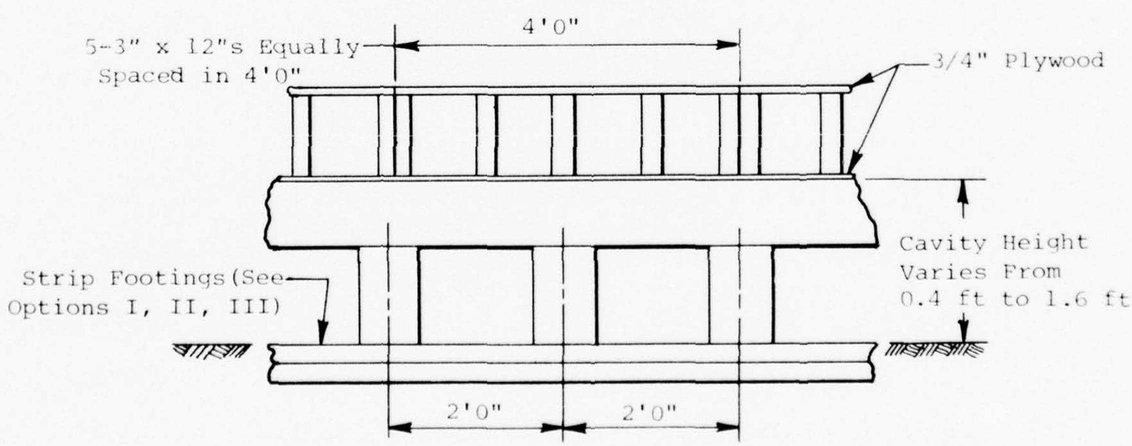
a. Concept 1 - Wood Box and Girder Construction

This structural concept is similar to that used in Events 2B and 3 of the HARD PAN I series and variations that have been used extensively in other HEST tests. Thus, a high degree of confidence in achieving the desired simulation accuracy would be expected.

The surcharge is supported by prefabricated wood boxes which are in turn supported by wood girders, columns and footings (fig. 5). The boxes are made 4 x 8 or 4 x 16 feet to utilize standard size sheets of plywood. Nominal 8 x 8-inch girders are placed on 8-foot centers and supported by 8 x 8-inch columns placed 2 feet on centers. The continuous footing supporting the columns consists of a single 3 x 16-inch plank resting on two 3 x 12-inch planks. The modular nature of this concept allows prefabrication and precutting of all structural elements prior to placement in the test facility.

The footings, columns and girders are set in position and the explosive charges placed. The prefabricated wood boxes minus the top plywood cover are then placed on the supporting girders and the open boxes filled with surcharge material. The plywood cover is attached and the remaining required depth of surcharge placed. There is no cutting or nailing in close proximity to the explosives placed in the cavity. The wood boxes provide a flat ceiling and columns and girders are the only structural elements projecting into the HEST cavity.





Section A-A

Figure 5. Wood Box and Girder Concept

Except for the first 188 feet of the HEST structure at the high pressure end where the tapered cavity height is less than the depth of an 8 x 8-inch girder, cavity height is varied by adjusting the height of the supporting columns. In the first 188 feet, other techniques must be used. Three possible options are shown in figure 6. Option I would require tapering the girders. The tapered girder would rest on and be fastened to the top of the wood footing. The tops of the footings would be placed at final test bed elevation where they could serve as screeds for finish grading of the test bed. Option II maintains a constant girder depth and slope but would require burying the footings beneath the floor of the test bed. Option III would vary cavity height in steps as opposed to the continuous increase provided by options I and II. Incremental increases as small as 0.25 inch are possible using standard thicknesses of timber and plywood. The option III girders would also rest directly on the top surface of the wood footings which would be placed at final test bed elevation.

Over the first 188 feet of the test bed, the three options for varying the cavity height have the disadvantage of increased difficulty in making cross ties between the rows of charges placed between girders. For the first 188 feet, holes must be drilled through the girders for the ties. Beyond this distance, the bottom of the girders will be above the floor of the test bed.

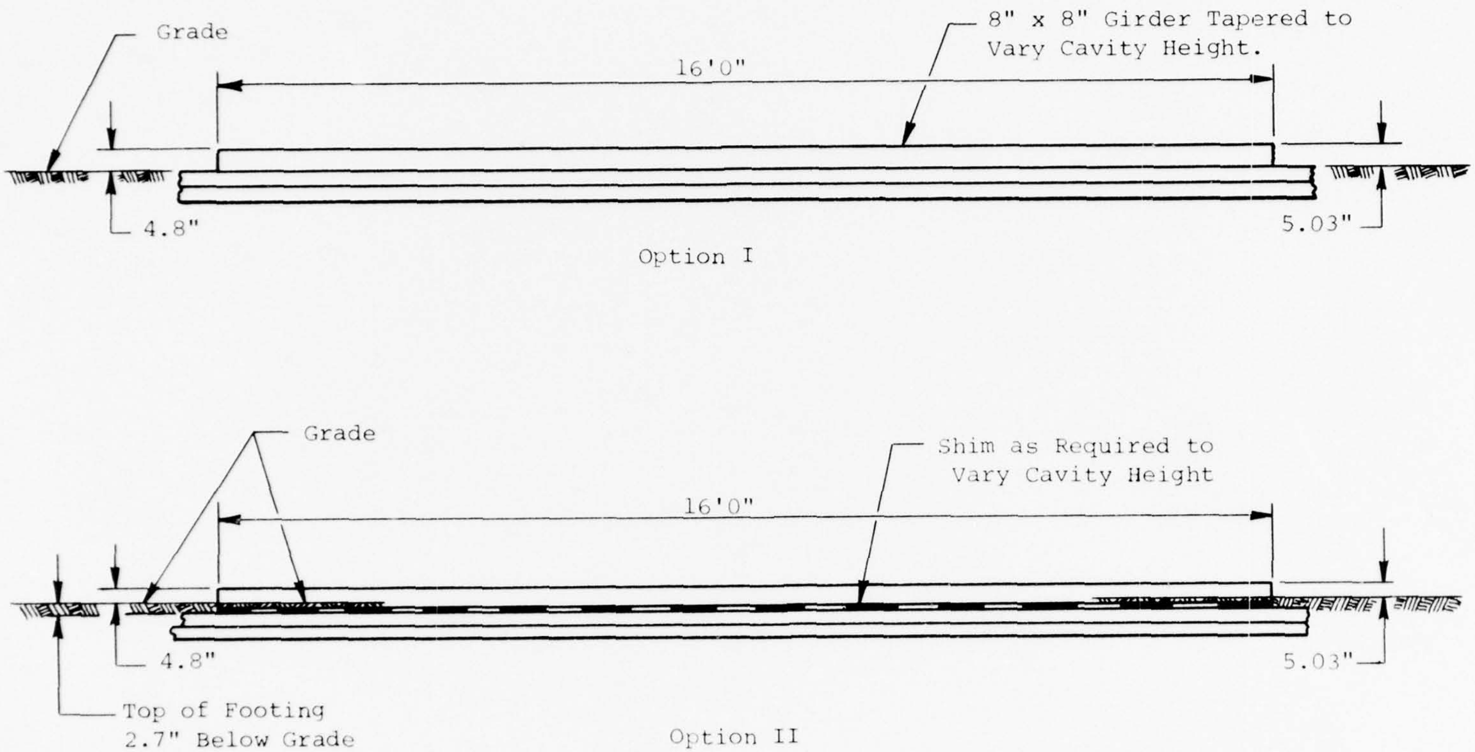
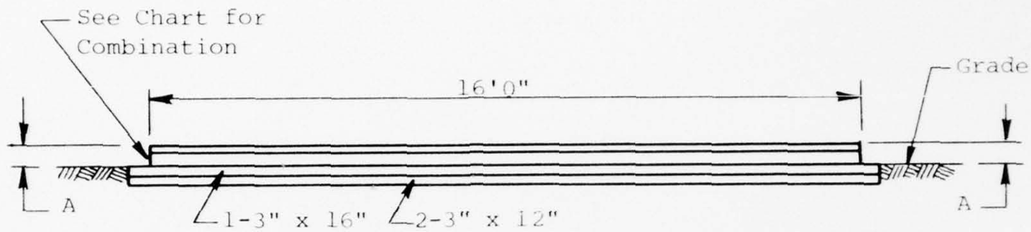


Figure 6a. Schemes to Adjust Cavity Height



Option III

DIM A	THICKNESS COMBINATION
5.0	1-4 x 8 + 1-2 x 8
5.25	2-3 x 8 + 1/4" Plywood
5.5	1-6 x 8
5.75	1-6 x 8 + 1/4" Plywood
6.0	1-6 x 8 + 1/2" Plywood
6.25	1-6 x 8 + 3/4" Plywood
6.5	2-3 x 8 + 1-2 x 8
6.75	1-4 x 8 + 1-3 x 8 + 3/4" Plywood
7.0	1-6 x 8 + 1-2 x 8
7.25	1-6 x 8 + 1-2 x 8 + 1/4" Plywood
7.5	1-8 x 8

Figure 6b. Schemes to Adjust Cavity Height (Concluded)

A wood surcharge support structure results in considerable posttest debris of a size which must be removed by hand labor. Depending on the desired degree of cleanup during the site restoration phase, many of the smaller pieces of wood debris could simply be covered with soil and allowed to decay.

b. Concept 2 - Steel Roof Deck and Girder Construction

Concept 2 employs essentially the same girder, post and footing system as concept 1. However, it uses a steel deck instead of wood boxes to support the surcharge. This concept is shown in figure 7.

The steel decking is built up from Section 12-12 Q-deck units, 24 feet long by 12 inches wide. Support is provided by wood girders, columns and footings. The footings and girders are placed on 6-foot centers and supported by 8 x 8-inch columns spaced two feet nine inches on center. The footings are built up from one 3 x 12-inch plank placed atop two 3 x 10-inch planks. The girders are 8 x 8-inch sections.

The footings, columns and girders are erected and the explosives placed. The Q-decking is placed on the girders and fastened with screws. Nails may also be used if nailing is allowed after placement of explosives in the cavity. The Q-decking provides an irregular ceiling due to the shape of the deck section which must be considered in determining the effective depth of the cavity.

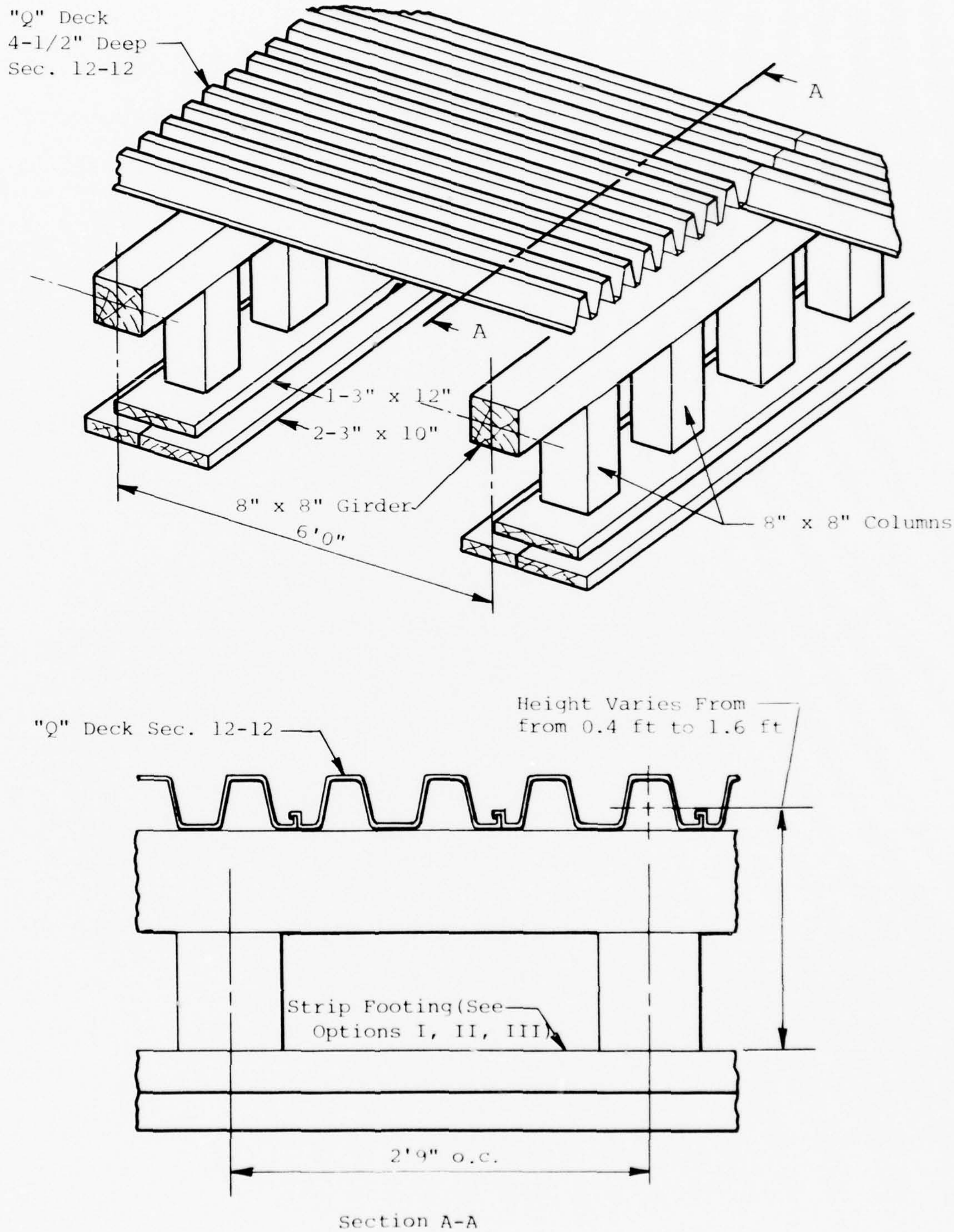


Figure 7. Steel Roof Deck and Girder Concept

As in concept 1, the cavity height is less than the depth of the 8 x 8-inch girders at the high pressure end of the cavity, and similar techniques are required to adjust the cavity height. Because of the irregular shape of the Q-deck, adjustment is required over the first 344 feet of the test bed.

c. Concept 3 - Concrete Panels and Posts

This concept makes maximum use of precast concrete sections. The general layout consists of large precast drop panels supported by precast concrete posts as shown in figure 8. A limited cost minimization study was conducted in connection with this concept. Four precast concrete concepts were compared on the basis of cost per foot of the test bed. Included were a uniform thickness slab, a ribbed slab, and two column spacings for slabs with drop panels. The most economical concept was the slab with drop panels for a column spacing of four feet on center in each direction. The panels are made as large as practical to reduce installation costs. The continuous strip footings are cast-in-place. The supporting columns are hand set by turning them on to anchor bolts set in the footings. Panel installation uses standard construction techniques. The weight of a single 24 x 24-foot panel is approximately 11 tons.

The assembly sketch depicted in figure 9 is an elevation view through the centerline of a typical post. The

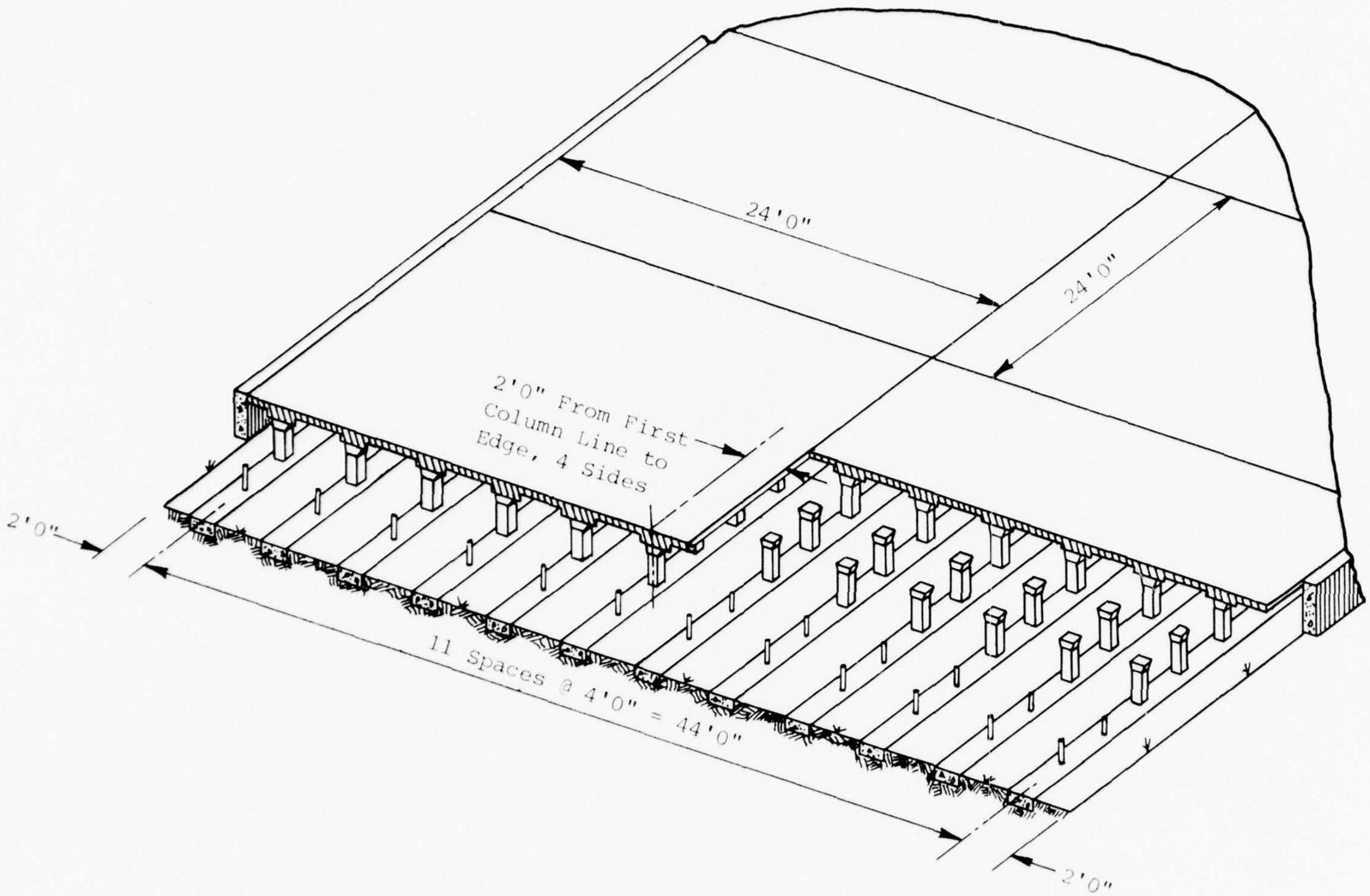


Figure 8. Concrete Panel and Post Concept

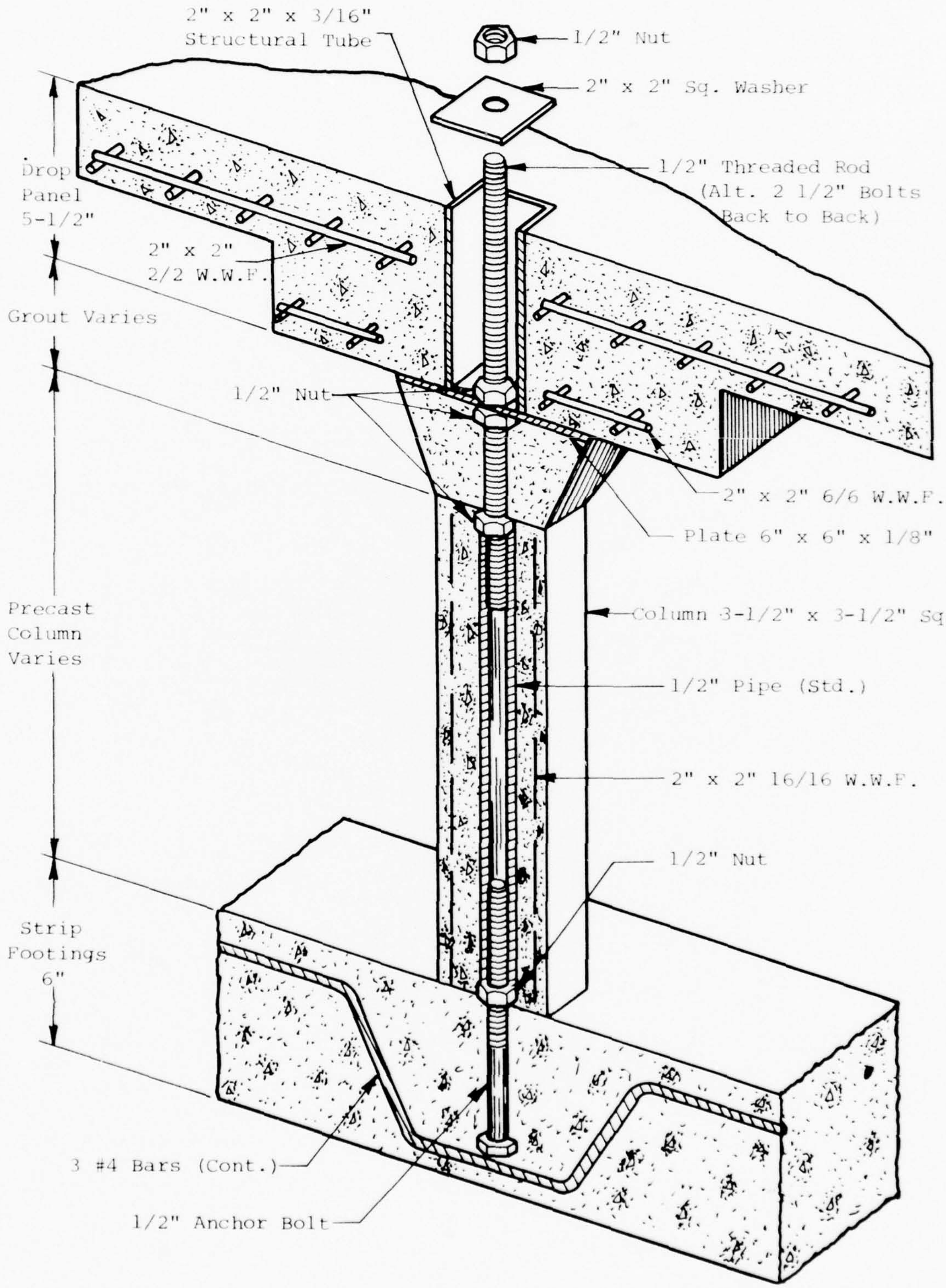


Figure 9. Concrete Panel and Post Assembly

top surface of the continuous strip footing is at the elevation of the surface of the test bed. Footings are 6 inches deep by 12 inches wide and spaced four feet on center. The footing is reinforced with three #4 bars for its full length. The post has a 0.5-inch standard steel pipe precast through its center with 0.5-inch diameter nuts welded to the pipe at its top and bottom ends. Half-inch diameter column anchor bolts are set in the footings on 4-foot centers. The columns are either 3.5 inches square or 4.25 inches in diameter and reinforced with 2 x 2-16/16 welded wire fabric. After the column is set in position, a cap plate is attached and double nutted to a 0.5-inch threaded rod set into the top nut of the post. The threaded rod is adjusted to set the cap plate at the desired elevation, and then grouted in place. After placement of the panel on the post, each rod is centered inside the structural tube and fastened by a washer and nut. The main panel reinforcement is 2 x 2-2/2 welded wire fabric centered in the 2-3/4-inch panel thickness. Each drop is reinforced with 2 x 2-6/6 welded wire fabric set 1-1/4 inches from its bottom surface. For the first 300 feet at the high pressure end of the test bed, columns are unnecessary and the panels can be placed directly on grout pads set around the anchor bolts. The elevation of the top surface of the grout pads can be more closely controlled by placing the grout after the test bed has been brought to the final grade.

The panels can be safely placed with a 25-ton crane. Approximately 80 panels are required for the 1000-foot long HEST facility. Surcharging can proceed immediately after the top nut has been tightened.

d. Concept 4 - Egg-Crate Concept

As noted, slurry or solid explosives will probably be used to develop the pressures required at the high pressure end of the MX trench test facility. Large clear areas are not required in this zone, provided accessibility is furnished for routing the timing and firing system. Boxes containing explosives can be placed directly on the test bed surface to support the overburden (fig. 10). (These boxes are similar to the surcharge support boxes used in HEST facilities.) Egg-crate type grids are used inside the boxes. This gridwork supports and distributes the overburden load uniformly onto the test bed surface while maintaining the required cavity depth. A modified gridwork is used with detonating cord.

The boxes are prefabricated using standard 4 x 8-foot or 4 x 16-foot sheets of plywood and structural grade lumber. Details of the construction are shown in figure 11. Sides and ends are fabricated from 1-inch plywood or 2-inch lumber. The inside of the box is a gridwork of square cells with vertical walls the same height as the sides and ends of the box. The bottom of the box and the inside gridwork are fabricated from 0.5-inch plywood. Grids are 16 inches on

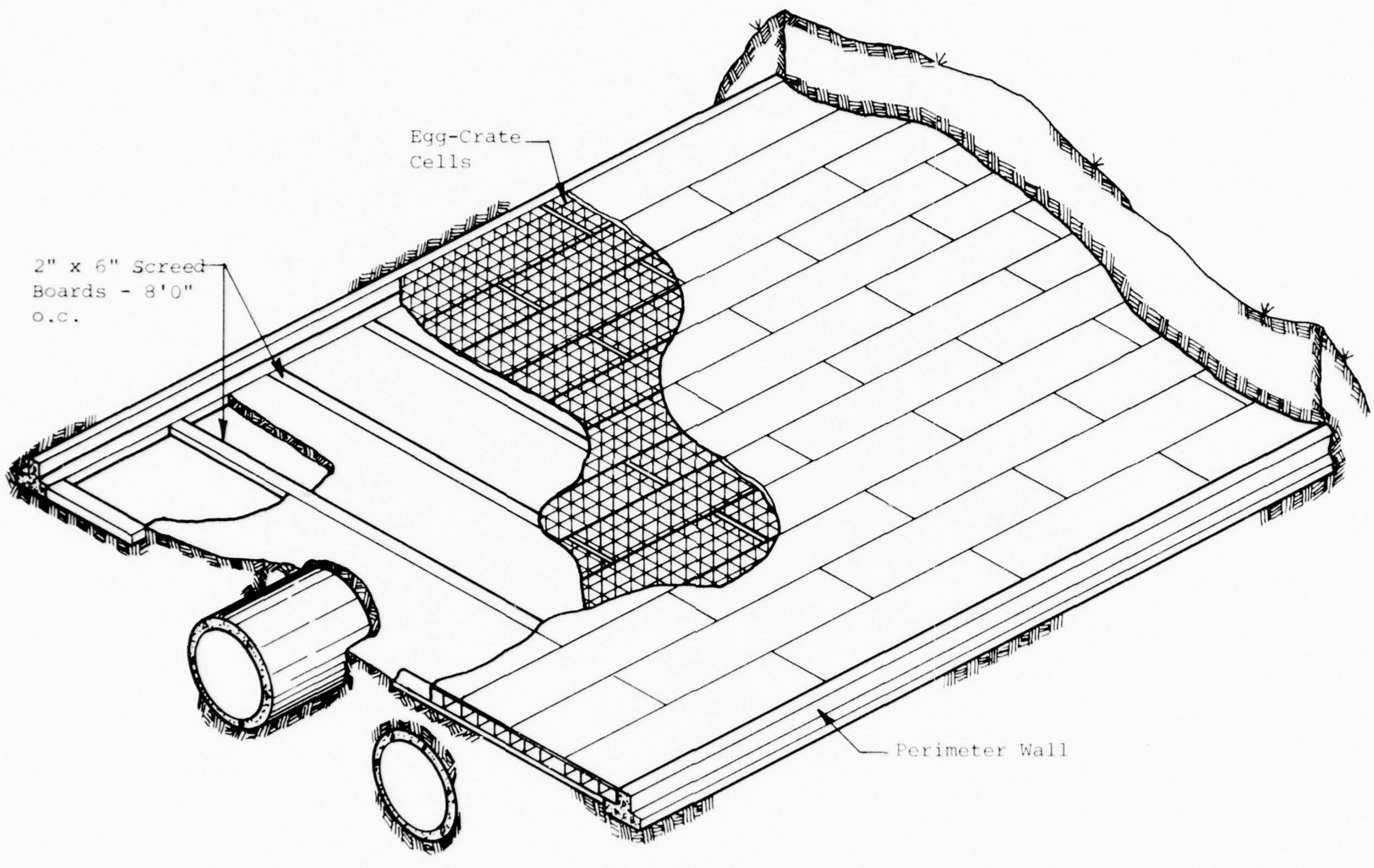


Figure 10. Egg-Crate Concept

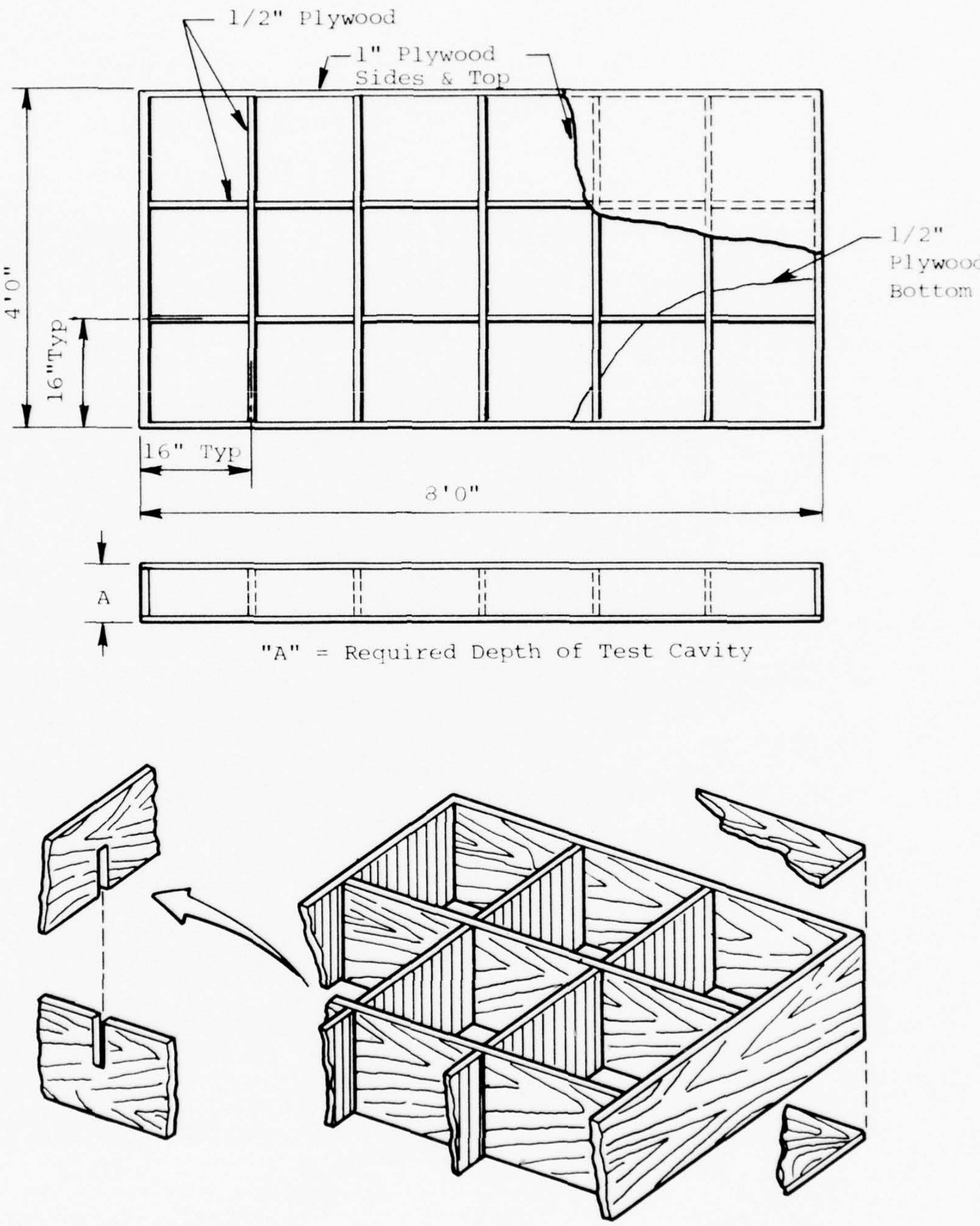


Figure 11. Egg-Crate Construction Details

center forming 18 cells in an 8-foot box. The sides and ends of the box are glued and/or nailed to the bottom, the grids are slotted and interconnected, then inserted and glued to the box along all interfaces. After installation in the test cavity, holes are notched or drilled through the cell walls for routing of the timing and firing system.

An alternate box design for use with detonating cord is similar to the egg-crate except that the end and transverse panels are not installed (fig. 12). Instead, 2 x 6-inch sections spaced on 16-inch centers are bonded to the top of the side and interior stringers of the box to provide adequate support for the top. Transverse spacers/stiffeners made from 2 x 6-inch lumber are fastened between stringers at the top and bottom of the box on 4-foot centers. Holes are notched or drilled through the side and interior stringers to interconnect the detonating cord weave at regular intervals. This design allows long continuous lengths of detonating cord to be installed in the test cavity. Lateral support is provided by the reinforced concrete wall around the perimeter of the test cavity.

Changes in cavity depth are made in several discrete steps along its length. For example, eight 2-inch steps would approximate the anticipated total depth variation over the 1000-foot test bed. Eight different box depths would be required. All boxes could be prefabricated at an on-site facility for subsequent installation in the test cavity.

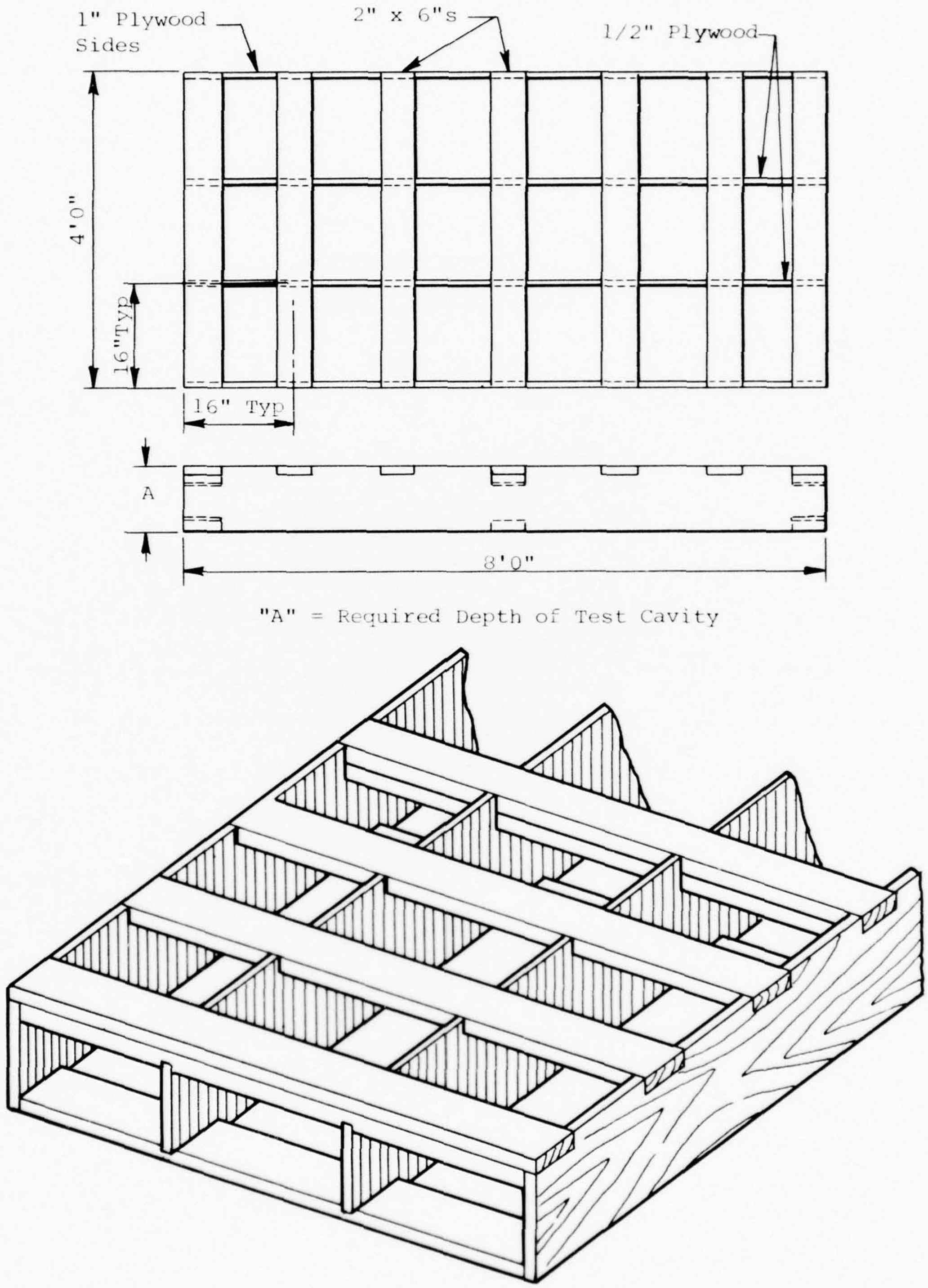


Figure 12. Alternate Egg-Crate Construction Details

The test bed surface can be brought to final grade using 2 x 6-inch screed boards. These boards are also used as bearing strips for initial alignment of the boxes. The next step is to place the explosives and timing and firing system. Finally, the 1-inch plywood tops are attached and the required overburden is added.

4. COST ESTIMATES

Cost estimates for each concept were based upon current material costs obtained from MEANS Building Construction Cost Data (1976), augmented by informal estimates obtained from local suppliers. Labor costs were derived using a reasonable estimate for the man hours needed to construct and install each of the concepts. The man-hour rates used for carpenters, laborers, rodmen, and other skills were subcontractor billing rates. Each cost estimate includes overhead, profit and a contingency.

Portions of the test facility that are common to all concepts such as the reinforced concrete wall surrounding the test bed, earth moving, and placement of the surcharge, are not included in the cost estimates. Thus, the costs presented reflect only the actual facility structural support system within the perimeter walls.

Estimates are given for each concept in terms of cost per linear foot of a 48-foot wide test bed.

a. Concept 1 - Wood Box and Girder Construction

Footings, Columns and Girders	
Material	$ 53
Labor	13
Boxes	
Material	149
Labor	25
	$240
30% O & P, Taxes	72
	$312
5% Contingency	16
TOTAL	$328 plf

b. Concept 2 - Steel Roof Deck and Girder Construction

Footings, Columns and Girders	
Material	$ 58
Labor	14
Q-Decking	
Material	156
Labor	58
	$286
30% O & P, Taxes	86
	$372
5% Contingency	19
TOTAL	$391 plf

c. Concept 3 - Concrete Panels and Posts

Panels	
Material	$ 72
Labor	43
Posts	
Material	10
Labor	48
Footings	
Material	24
Labor	17
Panel Installation	16
	$230
30% O & P, Taxes	69
	$299
5% Contingency	15
TOTAL	$314 plf

d. Concept 4 - Egg-Crate Concept

Egg-Crate	
Material	$ 90
Labor	22
	$112
30% O & P, Taxes	34
	$146
5% Contingency	8
TOTAL	$154 plf
Alternate Egg-Crate	
Material	$130
Labor	33
	$163
30% O & P, Taxes	49
	$212
5% Contingency	11
TOTAL	$223 plf

5. EVALUATION OF CONCEPTS

The advantages and disadvantages of each proposed concept are summarized below.

a. Concept 1 - Wood Box and Girder Construction

Advantages

- Proven concept
- Prefabricated components
- Minimizes safety hazards after explosives placement
- Readily available materials

Disadvantages

- Relatively large timber columns and girders required in HEST cavity
- Not easily adaptable to varying small cavity heights

b. Concept 2 - Steel Roof Deck and Girder Construction

Advantages

- Proven concept
- Prefabricated components
- Readily available materials

Disadvantages

- Irregular cavity ceiling in addition to larger timber columns and girders
- Not easily adaptable to varying small cavity heights

- Longer zone requiring cavity height/footing adjustment
- Posttest cleanup more difficult due to steel sections

c. Concept 3 - Concrete Panels and Posts

Advantages

- Maximum use of large precast panels cuts installation time
- Adaptable to small, varying cavity heights
- Minimum field fabrication
- Normally available materials can be used

Disadvantages

- *Difficult explosive suspension*
- More quality control required
- Concrete fallback creates difficult posttest cleanup

d. Concept 4 - Egg-Crate Concept

Advantages

- Provides excellent cavity volume control
- Minimizes settlement
- Provides good containment and control of discrete or diffuse slurry charges
- Prefabricated structural members

- Efficient use of structure by eliminating posts/girders/strip footings
- Installed using hand labor
- Eliminates in place framing operations
- Readily available materials

Disadvantages

- Unproven design - requires testing
- Gluing operations in on-site fabrication facility untried
- Less accessibility for detonating cord arrays

6. SUMMARY AND CONCLUSIONS

Each of the concepts presented meets the facility design criteria. Each is relatively compatible with the installation of the explosives and the timing and firing systems. Problems associated with the minimization of fallback are common to all. Therefore, cost was the controlling factor in the selection of a concept. On this basis, the egg-crate concept is recommended.

There are, however, qualifications to this recommendation. The gluing operations required for fabrication of the boxes were not attempted. Also, the structural integrity of the box was not physically tested. Although the boxes are expected to provide considerable reserve strength, they are dependent on lateral support from adjacent boxes and the concrete perimeter

wall. Therefore, it is recommended that field fabrication gluing operations be more thoroughly investigated and that a proof test of the egg-crate and alternate egg-crate boxes be conducted.

SECTION III

USE OF HEST FOR DYNAMIC AIRBLAST SIMULATION TESTS OF ABOVEGROUND STRUCTURES

1. GENERAL

Two specific areas must be examined in determining the feasibility of using the HEST concept to test the MX closed shelter. First, it must be determined if the conventional HEST facility is capable of simulating the total pressure loading expected on the structure. This loading includes the reflected and drag pressures associated with the incident overpressure of interest. Secondly, it must be determined if any major structural or installation problems would be encountered in building the HEST facility around a full-scale MX test structure of the shape and dimensions shown in figure 13. Two orientations of the structure with respect to the shock front are required, face-on and side-on.

2. AIRBLAST PRESSURE SIMULATION

An essential prerequisite to the feasibility evaluation is the derivation of the actual pressure and impulse time-histories that are to be simulated. The airblast pressure environment of interest is that which would be imposed upon a structure from a 1 MT surface burst with an incident over-pressure of 600 psi. Although the free-field behavior of an airblast shock wave can be adequately defined, some uncertainty

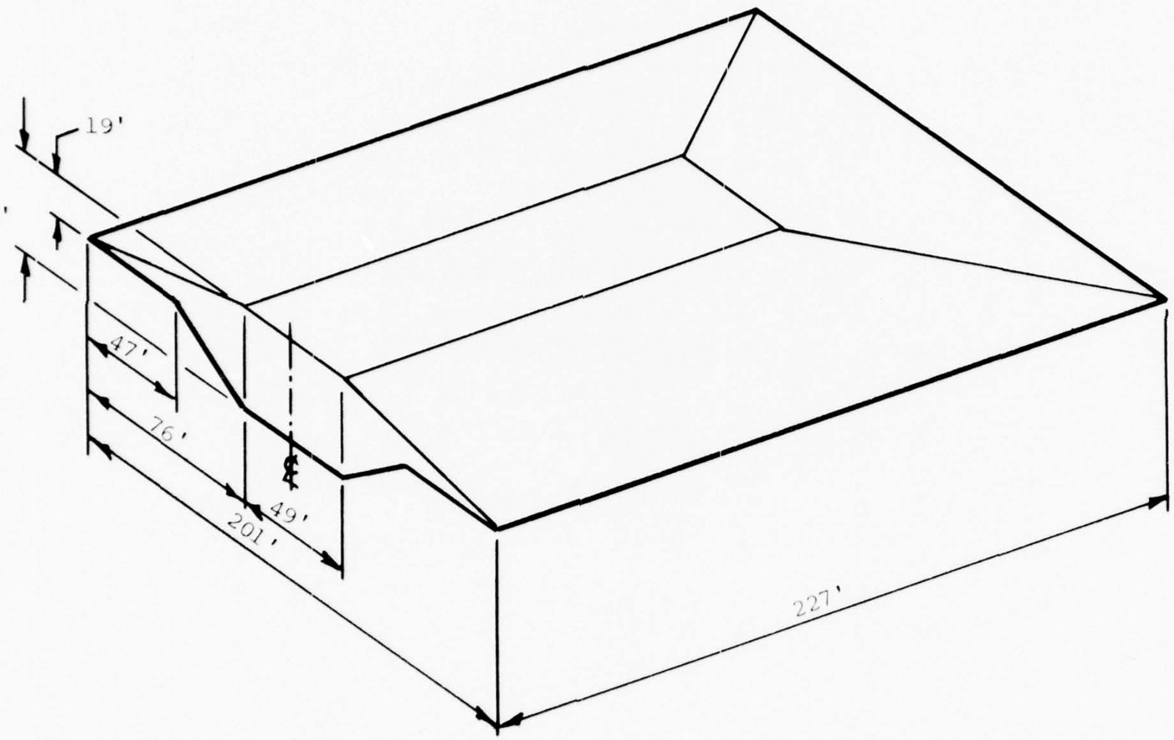


Figure 13. MX Test Structure

exists in regard to the airblast loading on various surfaces of an exposed structure.

As an airblast wave strikes the front face of an aboveground structure, a reflection occurs which momentarily increases the peak pressures to some level higher than the incident overpressure. Since the reflected pressure on the front face is greater than the incident pressure above and at the sides, it soon decays to a stagnation pressure which is the sum of the incident and the drag pressure. The decay time is approximately equal to the time required for a rarefaction wave to travel from the edge to the center of the face and return. As the shock wave continues, the sides, top and back surfaces of the structure are subjected to the incident pressure. At the same time, these surfaces are also subjected to dynamic pressures caused by transient airflow behind the shock front. This dynamic pressure induces drag loading on the structure, which may be positive or negative depending upon the orientation of the surface with respect to the shock front. The total loading on any face of the structure is the sum of the overpressure (including reflected pressure) and the drag pressure.

The detonation wave produced in a HEST facility is not a true shock wave, and the airblast phenomena associated with the detonation wave differ in various ways from that associated with the shock wave. Questionable dynamic (drag) pressure and only limited reflected pressure effects are achieved

by the HEST. Therefore, rather than applying an incident pressure wave to the structure and allowing its geometry to develop the desired reflected and drag pressure loading, the actual total pressure experienced by the structure must be applied directly by the HEST. Obviously, pressure time-histories for each surface of the test structure are required before a simulation can be attempted.

For the side-on condition where the test structure is oriented with its side facing the airblast shock front, total pressure data were obtained from several different sources. Pressure and impulse time-histories for some locations on the structure were available from earlier DABS I tests. A relatively complete set of pressure data was obtained from an AFTON computer calculation. Pressure profiles for each surface of the shelter were also constructed using the methods set forth in reference 1. Data from these sources were compared and evaluated. Reasonable agreement between scaled DABS I test data and the AFTON calculation was observed. Because of this agreement and the fact that it represented a fairly complete set of data, it was decided to use the results of the AFTON calculations as reference pressures for the HEST simulation investigation for the side-on orientation. Typical pressure and impulse curves from the AFTON calculations are shown in figures 14 and 15 for four locations on the leading

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1. Crawford, R.E., Higgins, C.J., and Bultmann, E.H., The Air Force Manual for Design and Analysis of Hardened Structures, AFWL TR 74-102, Air Force Weapons Laboratory, Kirtland AFB, N.M., October 1974.

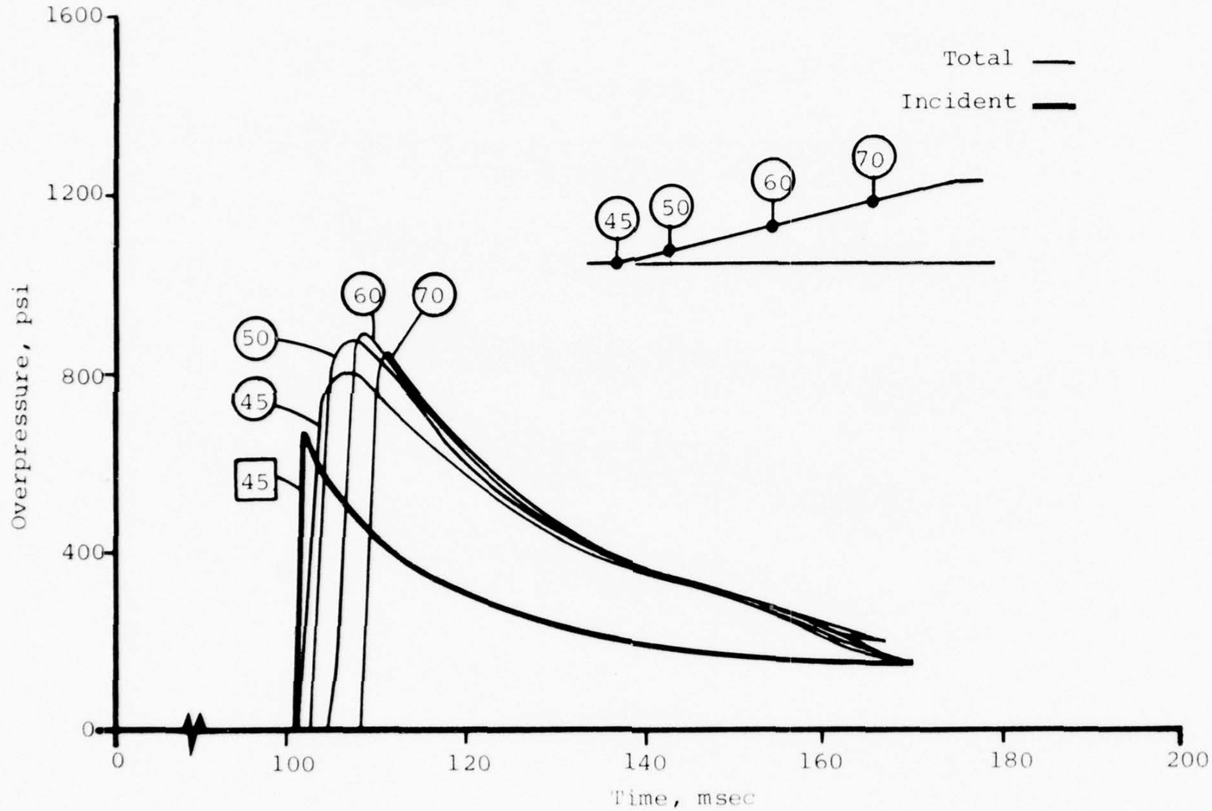


Figure 14. AFTON Pressure Time-Histories, Side-On Case

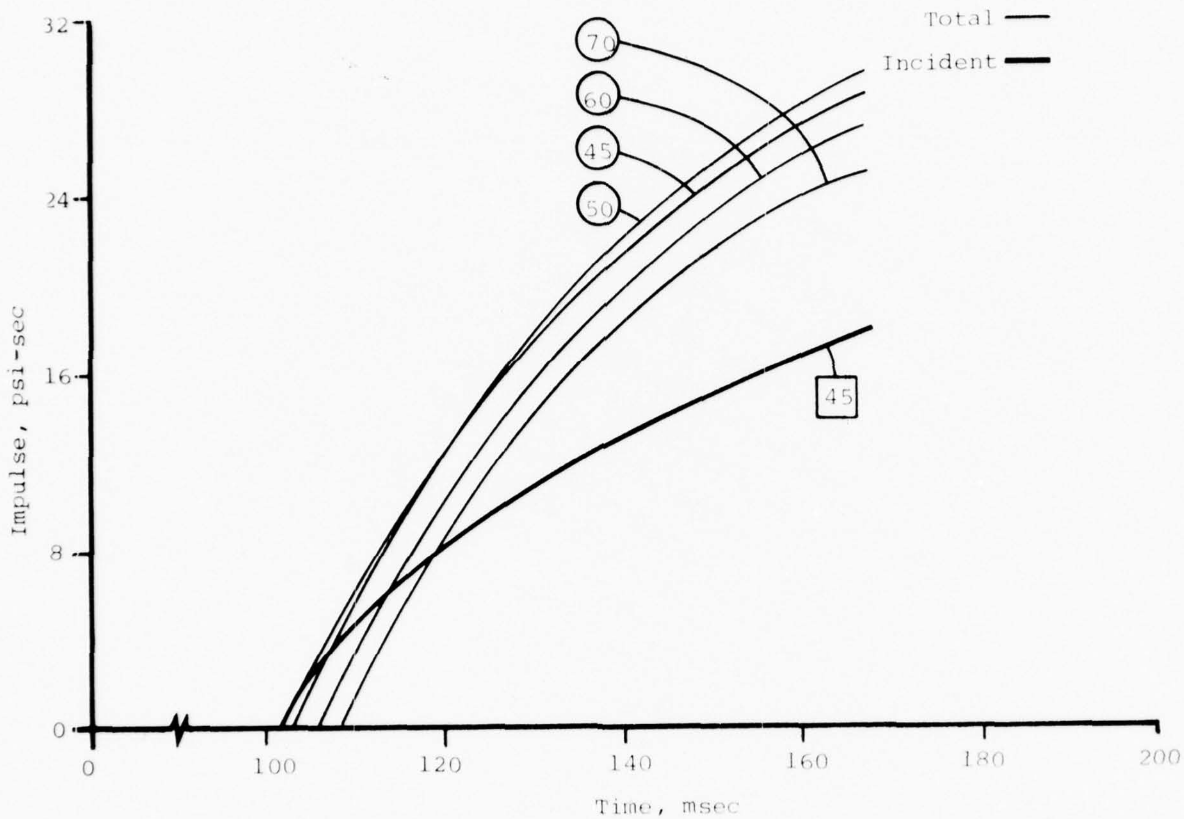


Figure 15. AFTON Impulse Time-Histories, Side-On Case

face of the structure. Also shown for comparison are the incident pressure and impulse at one location.

An unconfined explosion produces a pressure pulse of extremely short duration. In the HEST, the total pulse duration is increased, and the pressure decay behind the front is controlled to provide a realistic time varying pressure within the test cavity. This is accomplished by suitably adjusting the cavity depth and weight of overburden. These dimensions were determined using the Lock-Up Impulse Code developed at the Air Force Weapons Laboratory. The code calculates HEST pressure and impulse time-histories for given initial pressure, cavity and overburden dimensions, test bed and overburden soil properties, and the ratio of specific heats of the explosion products. Various combinations of cavity depth and overburden are assumed until the desired pressure and impulse are obtained. Initial calculations were made using soil densities of 95 pcf for the overburden and 100 pcf for the test bed. A specific heat ratio of 1.2 was used.

It was assumed that the HEST facility would be constructed directly over the test structure. To expedite the calculations, the facility was divided into four sections, and the cavity and overburden dimensions of each were determined separately. For the side-on case, the two sides, the top and the front of the test structure were examined individually. The tapering rear section of the shelter was treated as an extension of the

side and top surfaces. As a first approximation, only the reference pressure at the middle of each surface was applied to the entire surface. This resulted in each surface having a constant cavity depth and overburden, but the discontinuities between adjacent surfaces were quite large. A second iteration was made using reference pressures at other locations on the respective surfaces as input. It was found that the reference pressures matched closely and that the variable cavity depths are compatible between adjoining surfaces. To illustrate the agreement between the reference pressure and the pressure expected from a HEST test, loadings at two locations on the forward side of the structure are compared in figures 16 and 17. It is seen that the HEST pressure pulse can be made to closely match the expected total pressure pulse seen by the structure. Similar agreement was obtained for the top and rear sides.

The front surface of the test structure, which is parallel to the direction of propagation of the blast wave in the side-on orientation, sees only the incident pressure. Consequently, a uniform pressure equal to the free-field pressure obtained from the AFTON calculation would be applied to this surface. A vertical wall of soil is placed adjacent to the front face of the test structure at a distance equivalent to the cavity depth and with a thickness equal to the overburden height as determined from the Lock Up Code.

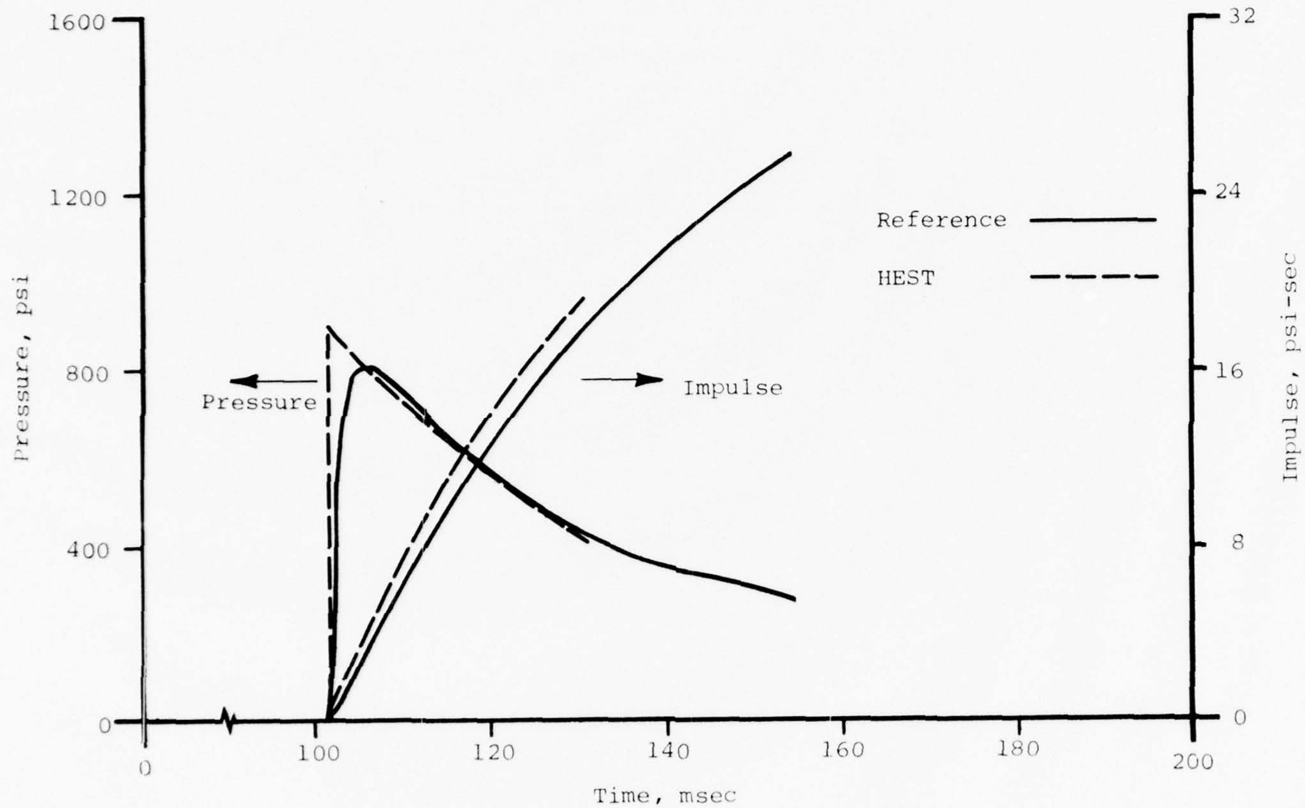


Figure 16. Comparison of HEST and Reference Pressure and Impulse - Station 45

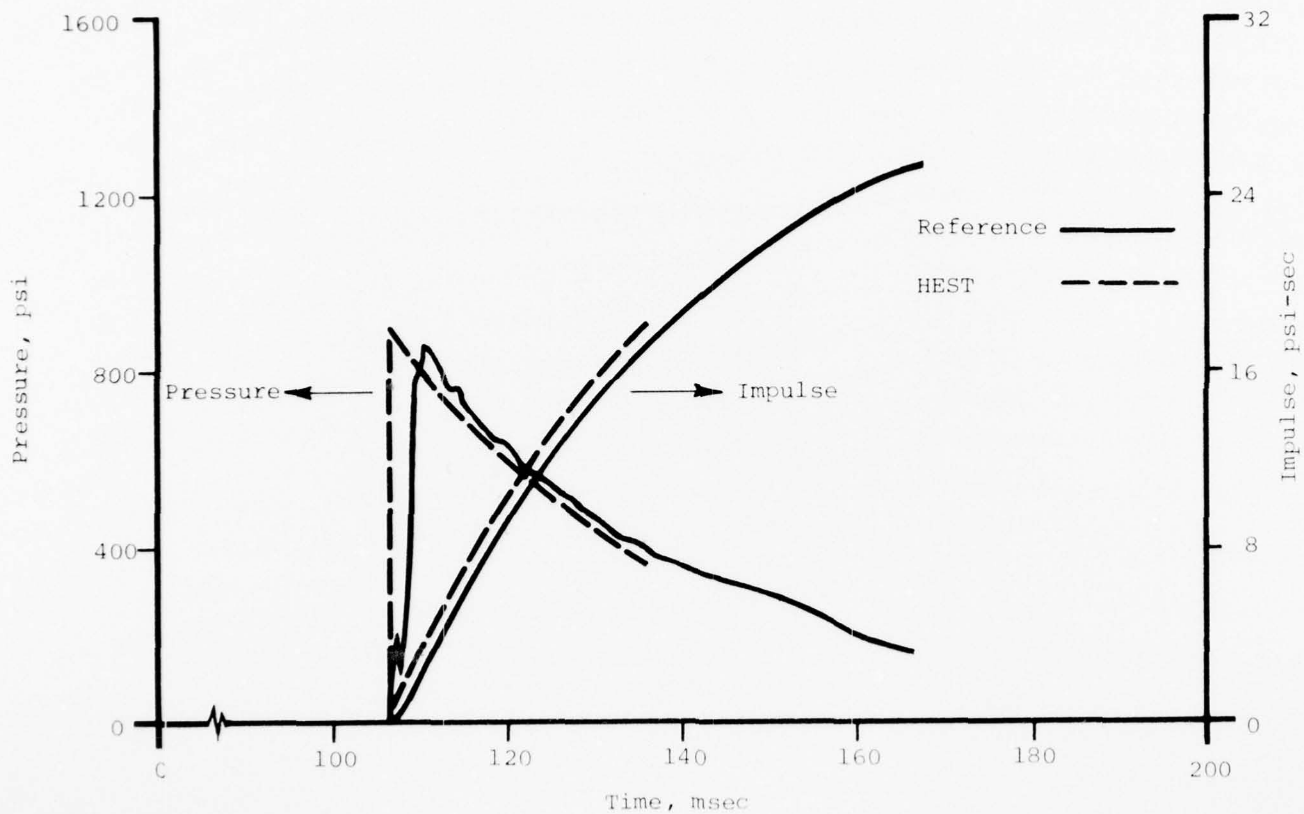


Figure 17. Comparison of HEST and Reference Pressure and Impulse - Station 70

A similar study was conducted for the same blast pressure environment and a face-on orientation of the structure. For this orientation, the proper simulation requires that the entire front face be loaded simultaneously by the blast wave which then envelopes the top and sides and, finally, the rear surface of the structure. The AFTON calculations for this condition were not available for use as a reference. On the front face, the reference pressure pulse was derived using the methods given in reference 1. The HEST and reference pressure and impulse time-histories on the front surface are compared in figure 18. As in the side-on case, the overburden for the front face is obtained through the use of a vertical wall of soil located parallel to that face and separated by a distance equivalent to the calculated cavity height.

Reference pressures on the top, side, and rear surfaces were approximated using the appropriate incident pressure time-history modified in accordance with the wave shapes obtained in the DABS 1C tests. A constant average pressure pulse was assumed to act over these surfaces. Therefore, no variation in cavity height or overburden depth is required for each of the surfaces. The reference pressures for all surfaces were again closely matched using the Lock-Up Code prediction technique.

Based upon these results and the past success of HEST tests in producing predicted pressure pulses, it appears feasible to use a HEST facility for simulation of dynamic airblast loading on aboveground structures.

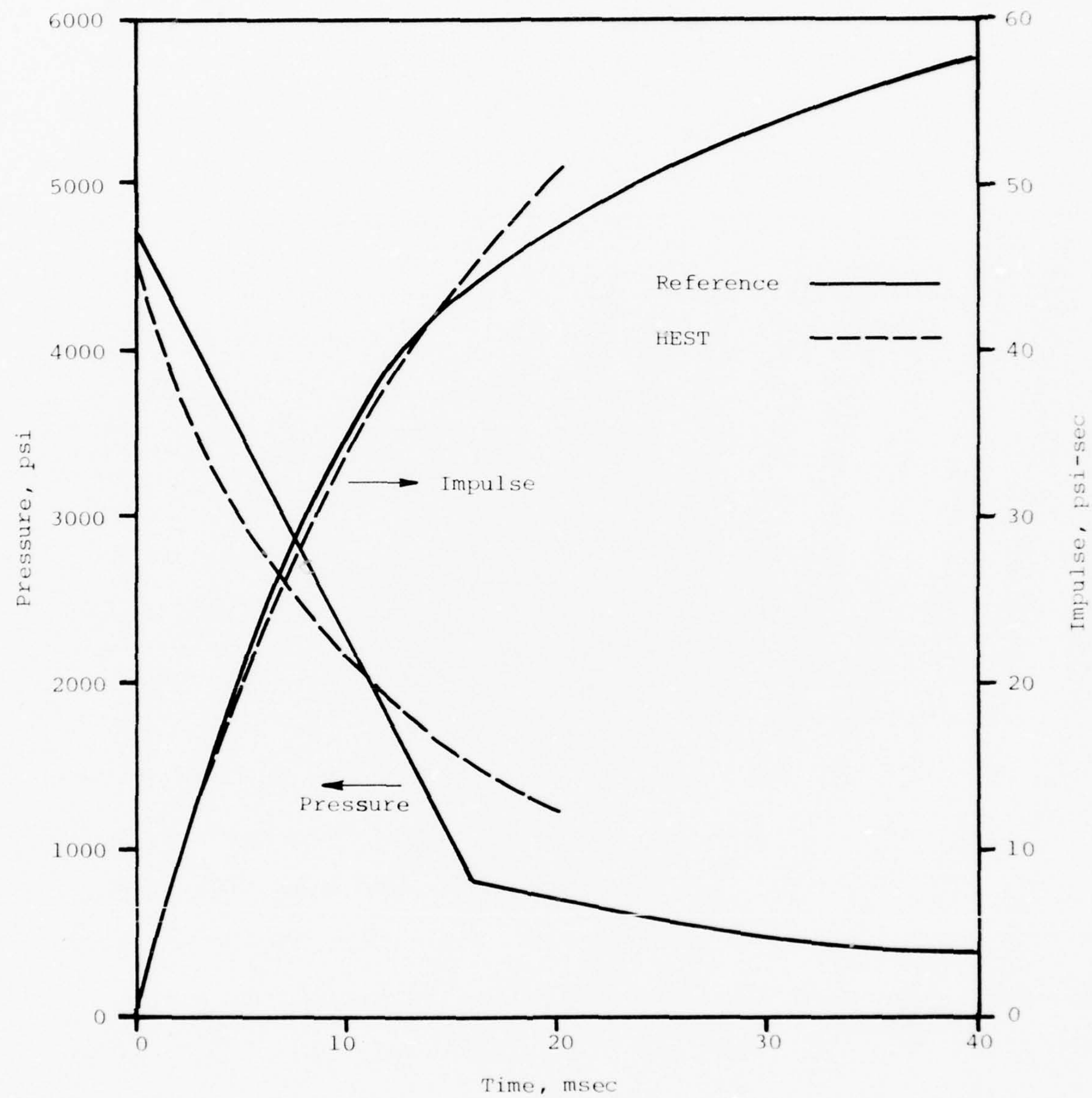


Figure 18. Comparison of HEST and Reference Pressure and Impulse-Front Surface, Face-On Case

3. PROPOSED HEST FACILITY

A HEST facility to test the MX closed shelter with simulated airblast loading would be built directly over the test structure. It would consist of a cavity of either uniform or varying height bounded on three sides by a reinforced concrete perimeter wall, above by the surcharge support structure, and below by the top, side, and rear surfaces of the structure to be tested. The cavity extends around the top surfaces to the front face of the structure, where it is bounded on the outside by a vertical wall of soil that is confined by a timber bulkhead.

The surcharge on top of the structure is supported by a system of prefabricated wood boxes, girders, columns and footings. The footings and girders are oriented in a direction parallel to the propagation of the blast wave. On the sloping surfaces, some cross bracing is required between the rows of girders. The perimeter wall must be designed to resist the horizontal reaction from the surcharge placed on the sloping surfaces.

For the side-on case shown in figure 19, 6 x 6-inch lumber is used for columns and girders. The girders are spaced on 8-foot centers, with the posts spaced on 2-foot centers. The continuous strip footing consists of two 3 x 16-inch planks placed atop one another. The support boxes are constructed from 0.75-inch plywood and five 2 x 10-inch joists on 1-foot

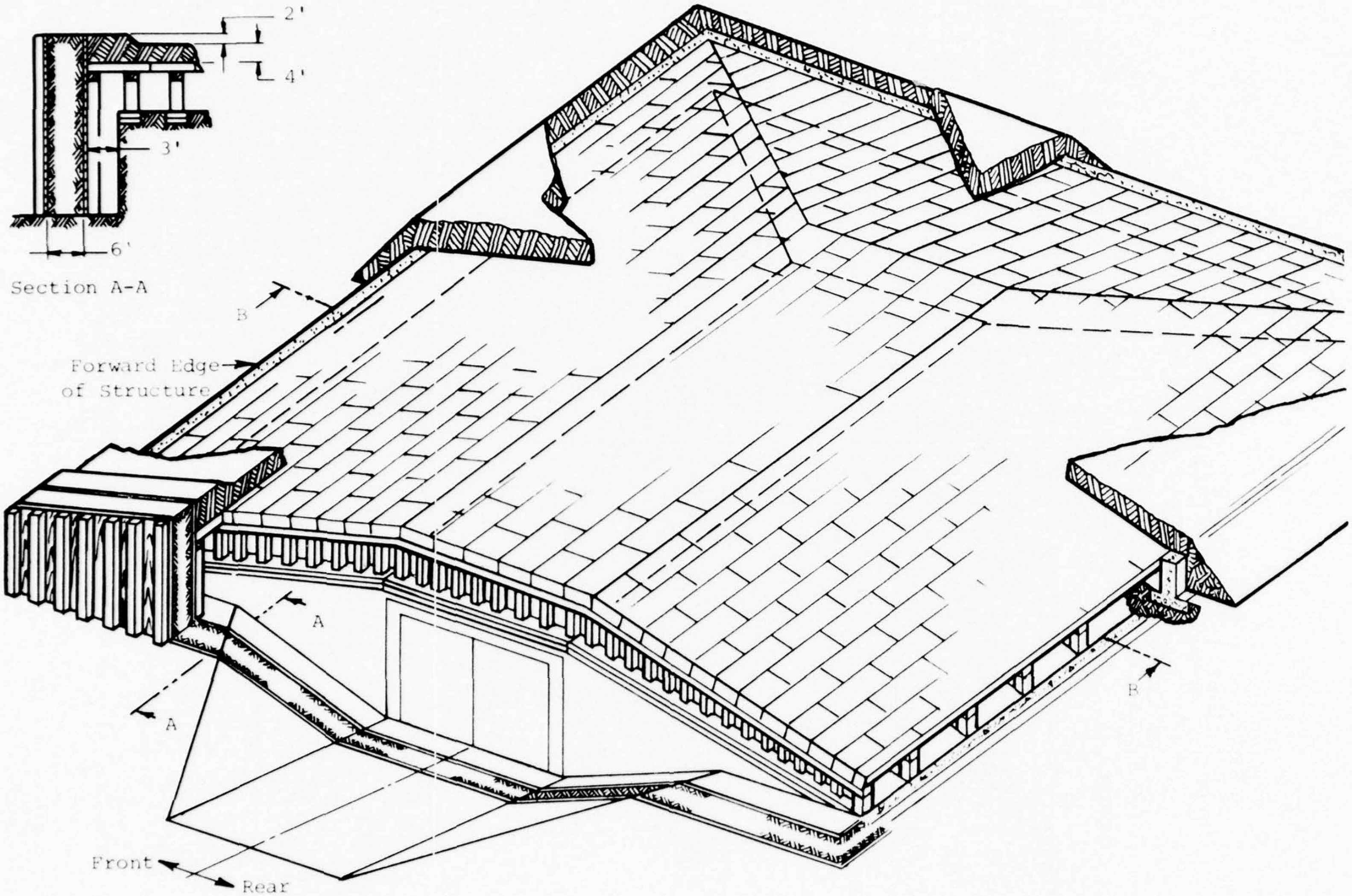


Figure 19a. HEST Test Facility, Side-On Case

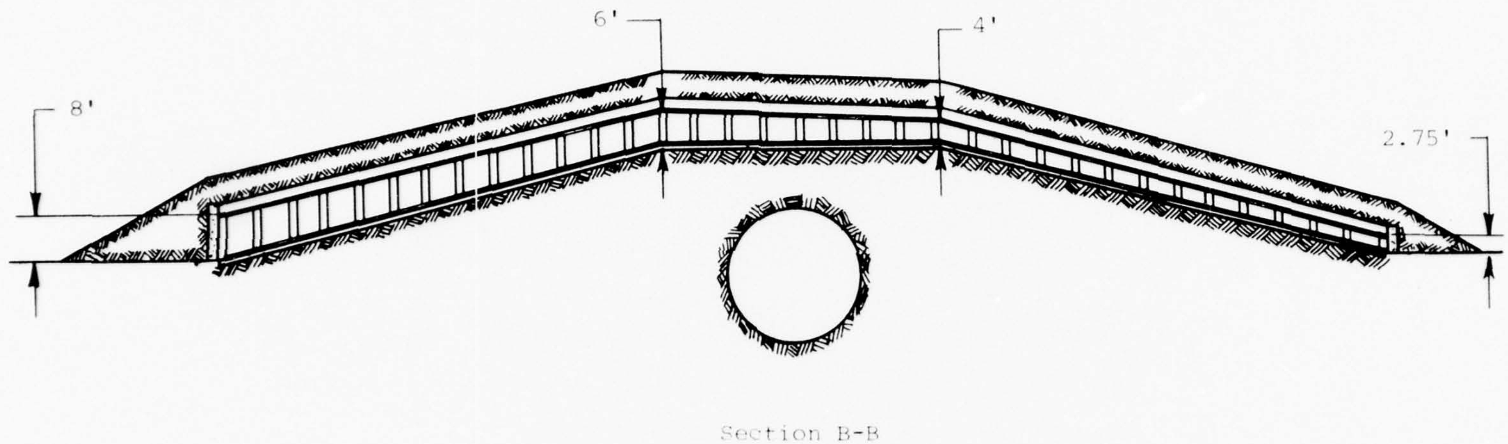


Figure 19b. HEST Test Facility, Side-On Case (concluded)

centers. The bulkheads for the vertical wall adjoining the front of the test structure are constructed of 0.75-inch plywood supported by 6 x 6-inch posts on 8-foot centers. Additional lateral restraint of the plywood walls is provided by 0.5-inch diameter stayrods on 1-foot centers. The wall is 6 feet thick and earth filled. The distance between the front face of the test structure and the inner surface of the wall is 3 feet.

In the side-on case, the cavity depth varies from 8 feet at the forward edge to 6 feet where the sloping front surface of the test structure joins the top surface. Along the top surface, the depth varies from 6 to 4 feet and then from 4 to 2.75 feet along the rear surface. Based on a soil density of 95 pounds per cubic foot, the depth of the surcharge is 4 feet along the front side and top. On the back side it is 3.75 feet deep.

For the face-on orientation of the structure, the same general construction is used. The columns and girders are 8 x 8-inch lumber. The footings are fabricated from a single 3 x 16-inch plank stacked on top of two 3 x 12 planks. The support boxes are made from 0.75-inch plywood with five 3 x 12-inch joists on 1-foot centers. The earth filled front wall is 6 feet thick. The bulkheads are constructed from 0.75-inch plywood with 4 x 4-inch posts on 4-foot centers. The plywood faces are additionally restrained by 0.5-inch

diameter stayrods on 1-foot centers. The distance between the inner surface of the front wall and face of the test structure is 8 feet. Over the entire structure the cavity depth is 3.5 feet and the surcharge depth is 8 feet.

4. COST ESTIMATE

The material quantities and total costs to construct a HEST facility over an existing full-scale MX closed shelter test structure are summarized in table 1. Both the side-on and face-on test facility costs are presented. Costs shown for each item include material, labor, overhead, profit and contingencies.

5. SUMMARY

The advantages and disadvantages of the HEST for testing the MX closed shelter are summarized below.

<u>Advantages</u>

- A full-scale test structure, including all sloping surfaces, may be tested.
- No major construction problems are anticipated, since construction methods similar to past HEST facilities are employed.
- Fallback should be minimal due to the sloping support structure and vertical front wall.
- Low cost.

Table 1

MATERIAL QUANTITIES AND COSTS FOR HEST FACILITIES

	Side-On		Face-On	
Item	Quantity	Cost ($)	Quantity	Cost ($)
Sawn Lumber	234,000 bf	219,375	296,000 bf	277,500
Plywood	110,000 sf	96,250	123,000 sf	107,625
Concrete	360 cy	72,000	250 cy	50,000
Overburden Placement	8,100 cy	20,250	15,500 cy	38,750
Site Rehabilitation	LS	10,000	LS	17,500
Total Cost		$417,875		$491,375

Disadvantages

- Total pressure rather than incident pressure environment must be predetermined and applied to the test structure.
- Actual degree of simulation is uncertain because of lack of prior test experience.

SECTION IV

IDENTIFICATION OF DABS FACILITY CONCEPTS

1. GENERAL

This section summarizes the results of a preliminary investigation of structural concepts that appeared capable of providing the necessary long, clear spans over the test section of a DABS facility. Since the intent was merely to evaluate the applicability of a concept and obtain relative cost information, only preliminary designs were prepared. These concepts could also be used over the driver section of the facility; however, some modifications to carry larger loads would be necessary. Structural concepts presented are designed to carry a surcharge load of either 1000 or 500 psf. Unless otherwise noted, an 8-foot spacing of the primary load carrying components is assumed to provide a common basis of comparison.

In concepts where the surcharge load is supported by spanning between the primary structural components, wooden boxes similar to those proposed for the high pressure HEST facilities are used. These boxes appear as decking in the cost estimates.

Retaining wall designs were based on the assumption that the wall height would be equal to 40 percent of the clear span of the structure. An exception to this is the arch with suspended platform, where the arch spans were increased to provide adequate clearance over the width of the test bed. Material quantity estimates for the retaining walls include the footings

for the walls. The design of the retaining walls is subject to significant change upon detailed investigation. It is also not clear that retaining walls are practical for the greater heights proposed. Depending on the soil properties at the site and a better definition of the minimum acceptable clear heights, more economical solutions might be possible.

Earthwork costs and the cost of endwalls are not included in the cost estimates since they are common factors to all concepts and would be essentially the same. Therefore, the cost estimates presented in this chapter represent only the basic cost of the structural system necessary to span the width of the test bed.

2. PARABOLIC ARCH WITH SUSPENDED PLATFORM

This concept utilizes parabolic concrete arches spaced 8 feet on center with retaining walls to obtain the required depth of the test section cavity. Spans of 60, 120, and 300 feet were investigated (figs. 20 through 22). A rise to span ratio of 0.25 was arbitrarily chosen and an arch cross-section selected based on the following items:

- Thrust at the crown.
- Thrust at the springline.
- Moment at the crown.
- Critical buckling load for an approximate arch.

Further optimization is possible. Presently, each arch shape must be braced to preclude out-of-plane instability.

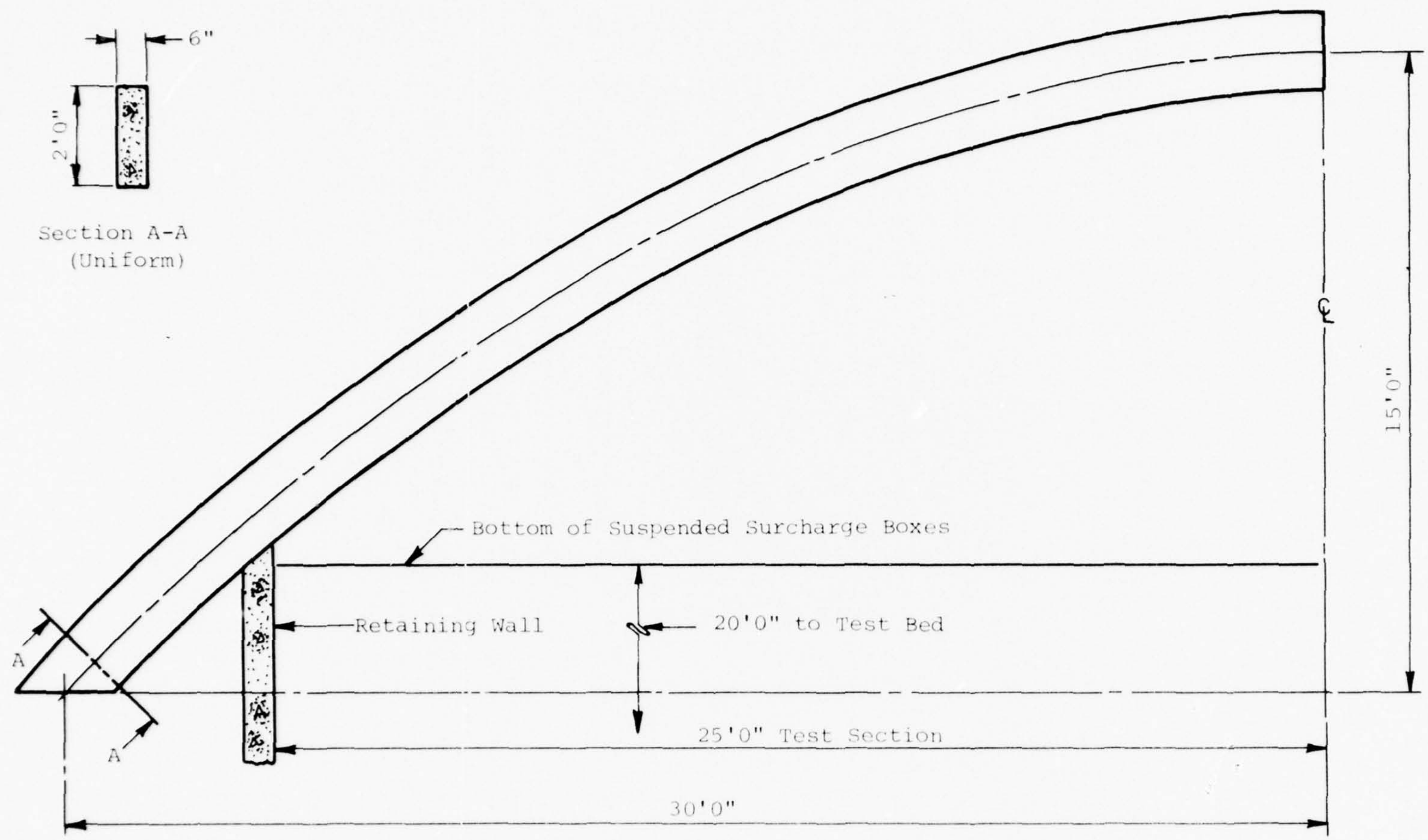


Figure 20. Parabolic Arch - 60-Foot Span

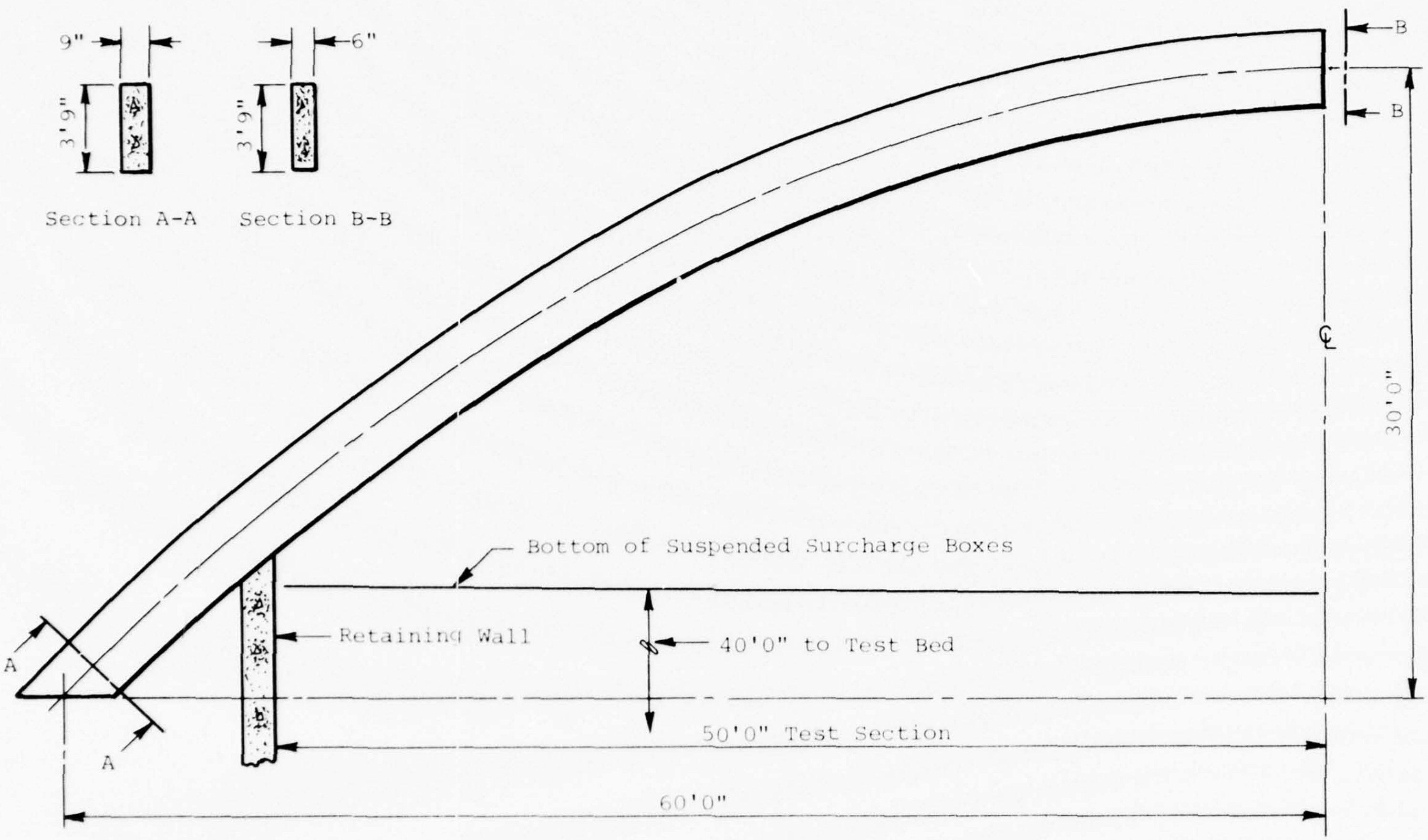


Figure 21. Parabolic Arch - 120-Foot Span

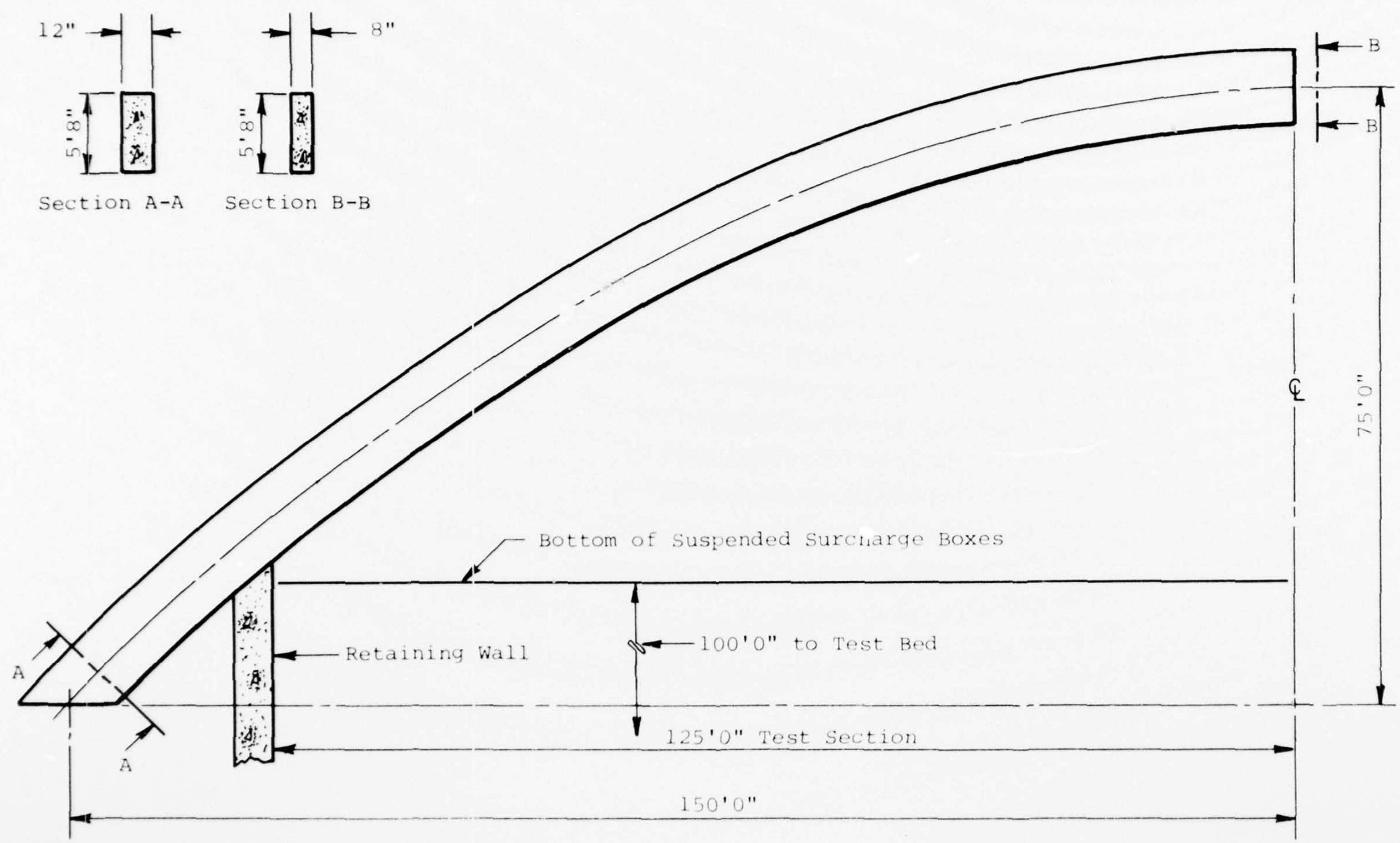


Figure 22. Parabolic Arch - 300-Foot Span

Each of the arches supports a hanger system, shown in figure 23, that is composed of back-to-back steel angles which, in turn, support a structural tee that is used to carry the 4 x 8-foot wooden boxes carrying the surcharge. The back-to-back angles are attached to hanger plates embedded in the concrete arch.

The arch may be cast-in-place or precast in sections and erected in the field. Form costs for the cast-in-place option may be more expensive than the precast installation for the longer spans because of the high crown elevation.

The material quantities and cost estimates per linear foot, excluding the arch footings, are

<u>60-Foot Span</u>

Arch Concrete	0.325	cy	@	$400	=	$ 130
Structural Steel	202	lbs	@	2	=	404
Decking	50	sf	@	6	=	300
20 Ft Retaining Wall	3.4	cy	@	400	=	1,360
						$ 2,194 plf

<u>120-Foot Span</u>

Arch Concrete	1.52	cy	@	$400	=	$ 608
Structural Steel	556	lbs	@	2	=	1,112
Decking	100	sf	@	6	=	600
40 Ft Retaining Wall	13.2	cy	@	400	=	5,280
						$ 7,600 plf

<u>300-Foot Span</u>

Arch Concrete	7.66	cy	@	$400	=	$ 3,064
Structural Steel	2535	lbs	@	2	=	5,070
Decking	250	sf	@	6	=	1,500
100 Ft Retaining Wall	78	cy	@	400	=	31,200
						$40,834 plf

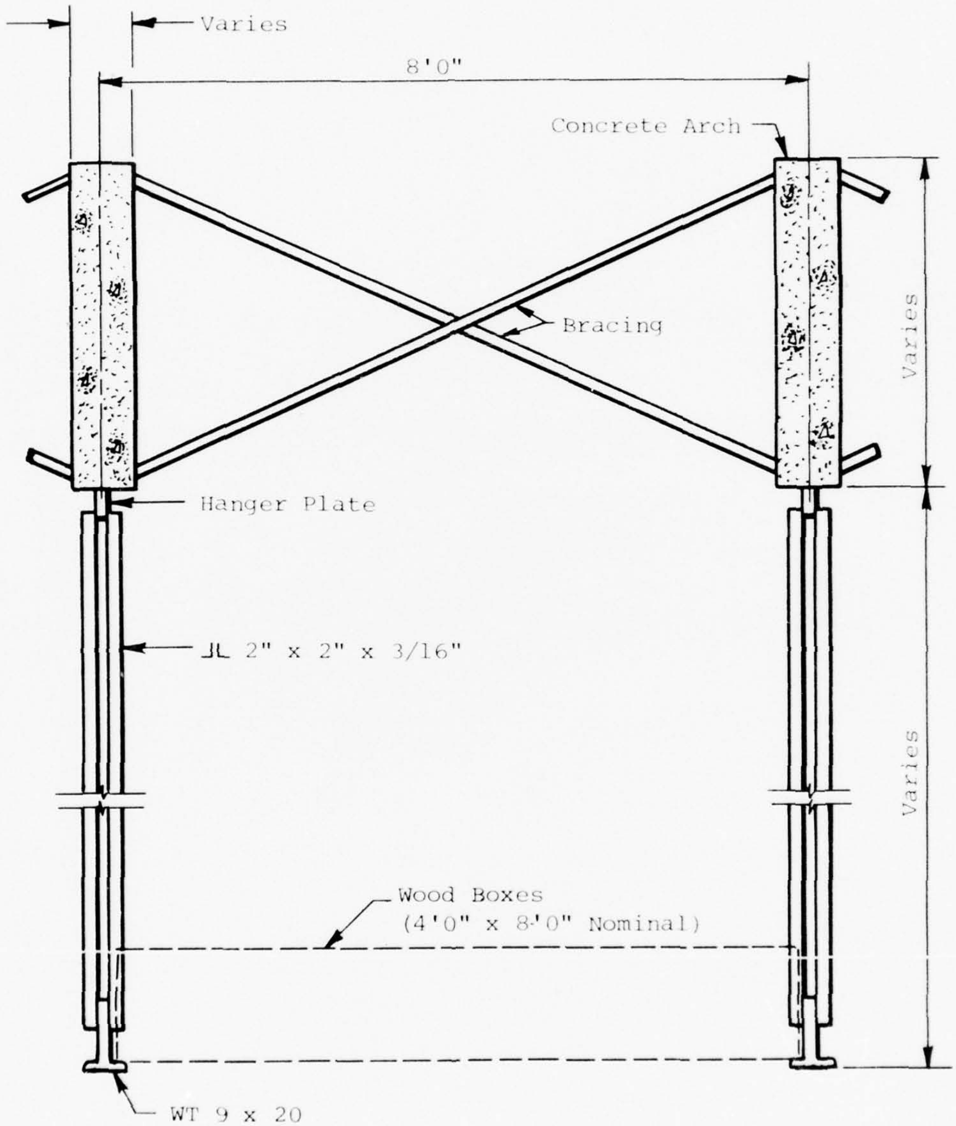


Figure 23. Parabolic Arch Hanger System

3. CABLE SUSPENSION SYSTEM

The parabolic cable suspension system concept utilizes a hanging cable supported by two towers. These towers are stabilized by support cables anchored in the ground. This concept is believed to be economical for the longer spans only. Therefore, only the 100 and 250-foot spans, figures 24 and 25, respectively, were estimated. The Class A Coated Bridge Strand parabolic cable supports vertical cables which support a floor grillage composed of steel shapes. Wood boxes placed on the grillage carry the surcharge. Rectangular steel plate sections are used for the towers.

The tower loads will require large footings. Special care will have to be exercised near the edges of the test bed to preclude slope instability. Construction of the suspension system towers and floor grillage involves only standard structural steel erection, but special crews and equipment may be required to erect the cable system.

The material quantities and cost estimates per linear foot, excluding the tower foundations and support cable anchors, are

<u>100-Foot Span</u>

Towers	2,350	lbs	@	$ 2	=	$ 4,700
Cables	520	lbs	@	4	=	2,080
Floor Grillage	840	lbs	@	2	=	1,680
Decking	100	sf	@	5	=	500
40 Ft Retaining Wall	13.2	cy	@	400	=	5,280
						$14,240 plf

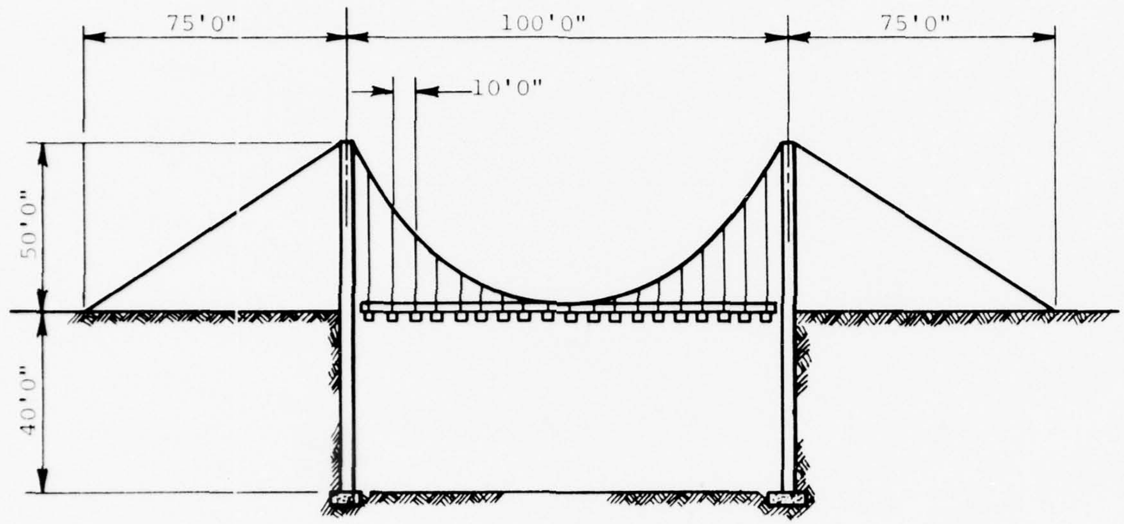


Figure 24. Cable Suspension System - 100-Foot Span

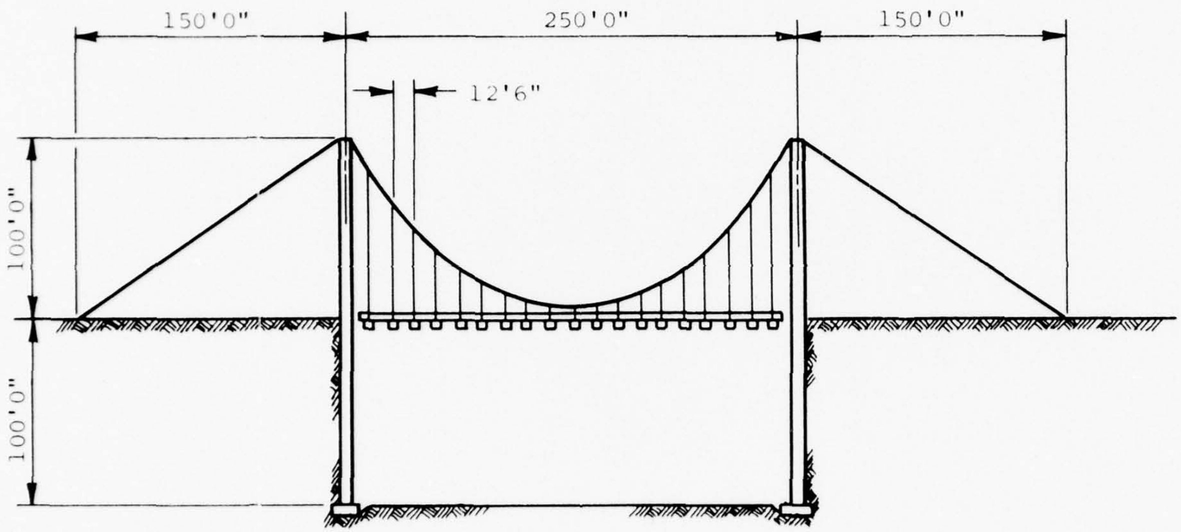


Figure 25. Cable Suspension System - 250-Foot Span

250-Foot Span

Towers	12,150 lbs	@	$ 2	=	$24,300	
Cables	3,025 lbs	@	4	=	12,100	
Floor Grillage	2,575 lbs	@	2	=	5,150	
Decking	250 sf	@	5	=	1,250	
100 Ft Retaining Wall	78 cy	@	400	=	31,200	
					$74,000	plf

4. CONCRETE BOX GIRDERS

This concept utilizes reinforced concrete box girders to carry the loads over the designated spans. Designs for spans of 50, 100 and 250 feet are shown in figures 26 through 28. The sections are heavily reinforced, and in an attempt to reduce the dead weight of the structures, the material strengths were increased in rough proportion to the spans.

The box sections will require foundations and retaining walls similar to those used for freeway bridge construction. Since the box girders form a continuous surface, no decking is required to carry the surcharge.

The material quantities and cost estimates per linear foot are

50-Foot Span

Box Girder Concrete	1.6 cy	@	$400	=	$ 640	
20 Ft Retaining Wall	3.4 cy	@	400	=	1,360	
					$ 2,000	plf

100-Foot Span

Box Girder Concrete	6.8 cy	@	$400	=	$ 2,720	
40 Ft Retaining Wall	13.2 cy	@	400	=	5,280	
					$ 8,000	plf

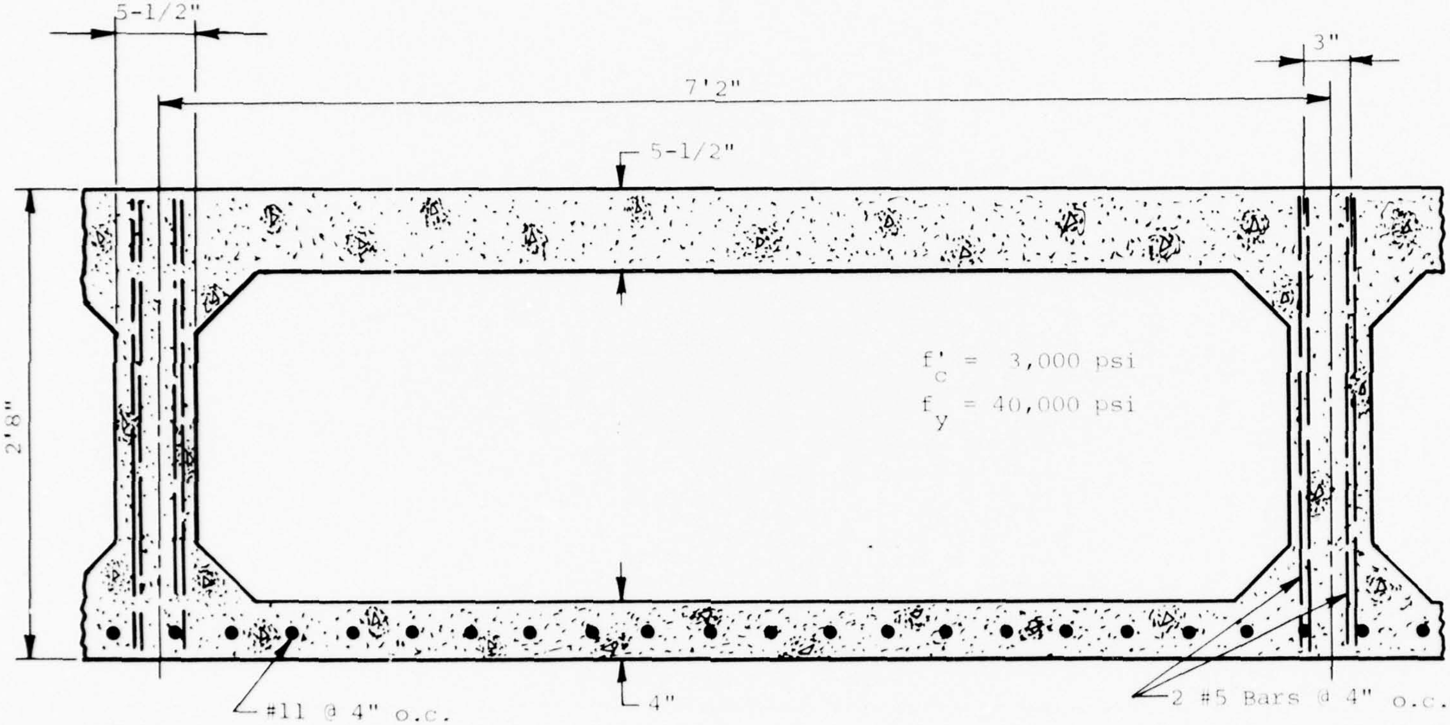


Figure 26. Box Girder Concept - 50-Foot Span

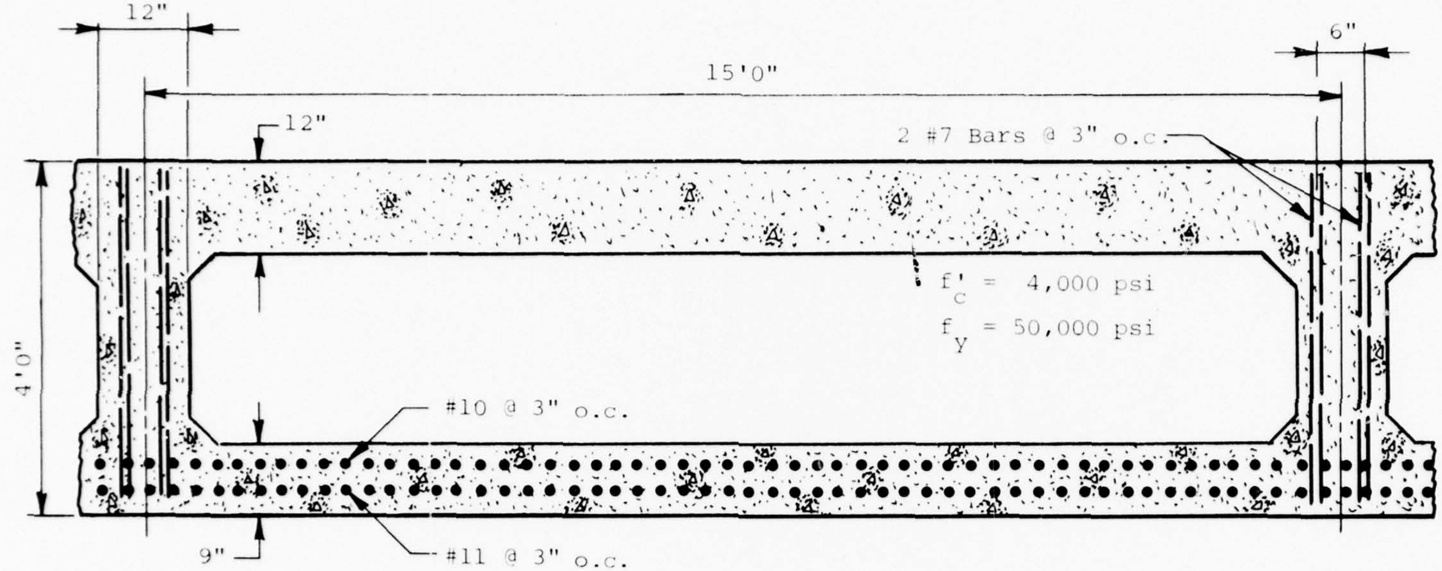


Figure 27. Box Girder Concept - 100-Foot Span

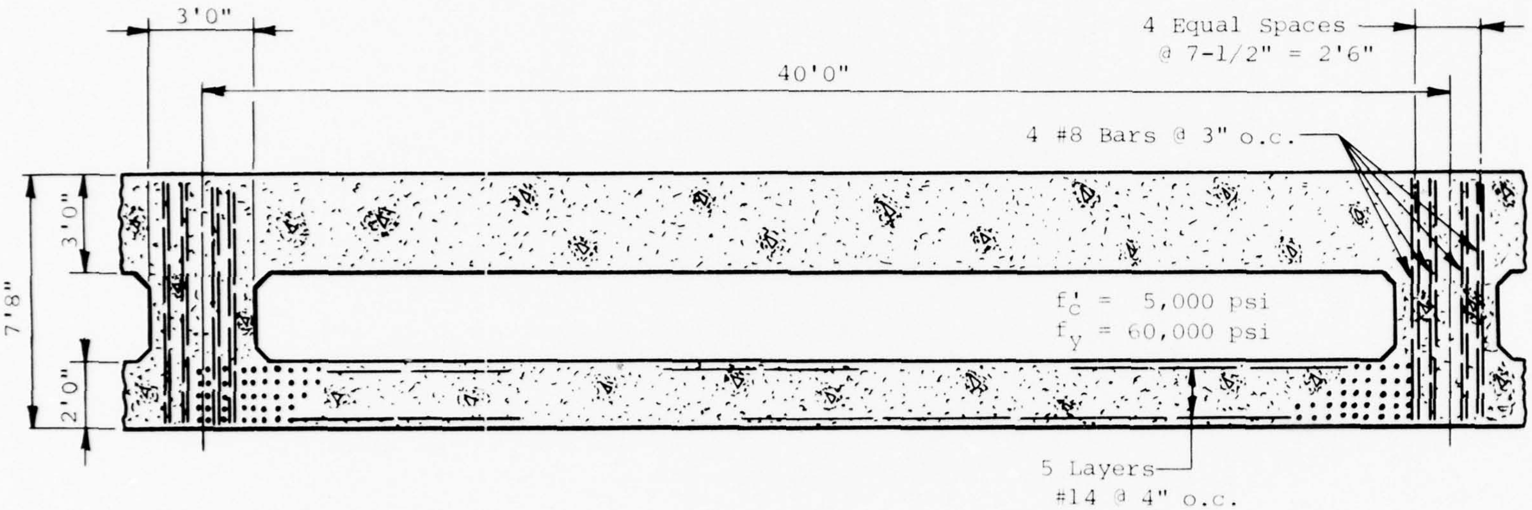


Figure 28. Box Girder Concept - 250-Foot Span

250-Foot Span

Box Girder Concrete	48.2	cy @	$400	=	$19,280	
100 Ft Retaining Wall	78	cy @	400	=	31,200	
					$50,480	plf

5. LONG-SPAN STEEL JOISTS

The primary structural element in this concept is the preengineered, shop-fabricated open web trusses that are commercially available. These trusses appear to be useable up to spans of about 125 feet. Structural sections were selected to satisfy the requirements for clear spans of 50 and 100 feet (figs. 29 and 30). For these spans, a combination foundation-retaining wall at the sides of the test bed might be used to support the steel joists. Figure 31 shows a scheme for constructing a 250-foot DABS facility using internal and external supports.

The number of trusses required to support the soil overburden was minimized by increasing the load carrying capacity of the trusses by 1.5 times their rated total safe load. The resulting stresses in the truss members are expected to be nearly equal to stresses developed in the prescribed performance test for checking truss members. This design assumption may place undesirable restrictions on the methods by which overburden can be placed and may have to be relaxed. The consequences will have to be evaluated in a more detailed analysis.

The decking material is plywood or 2-inch tongue-and-groove decking, depending upon the truss spacing. Steel roofing

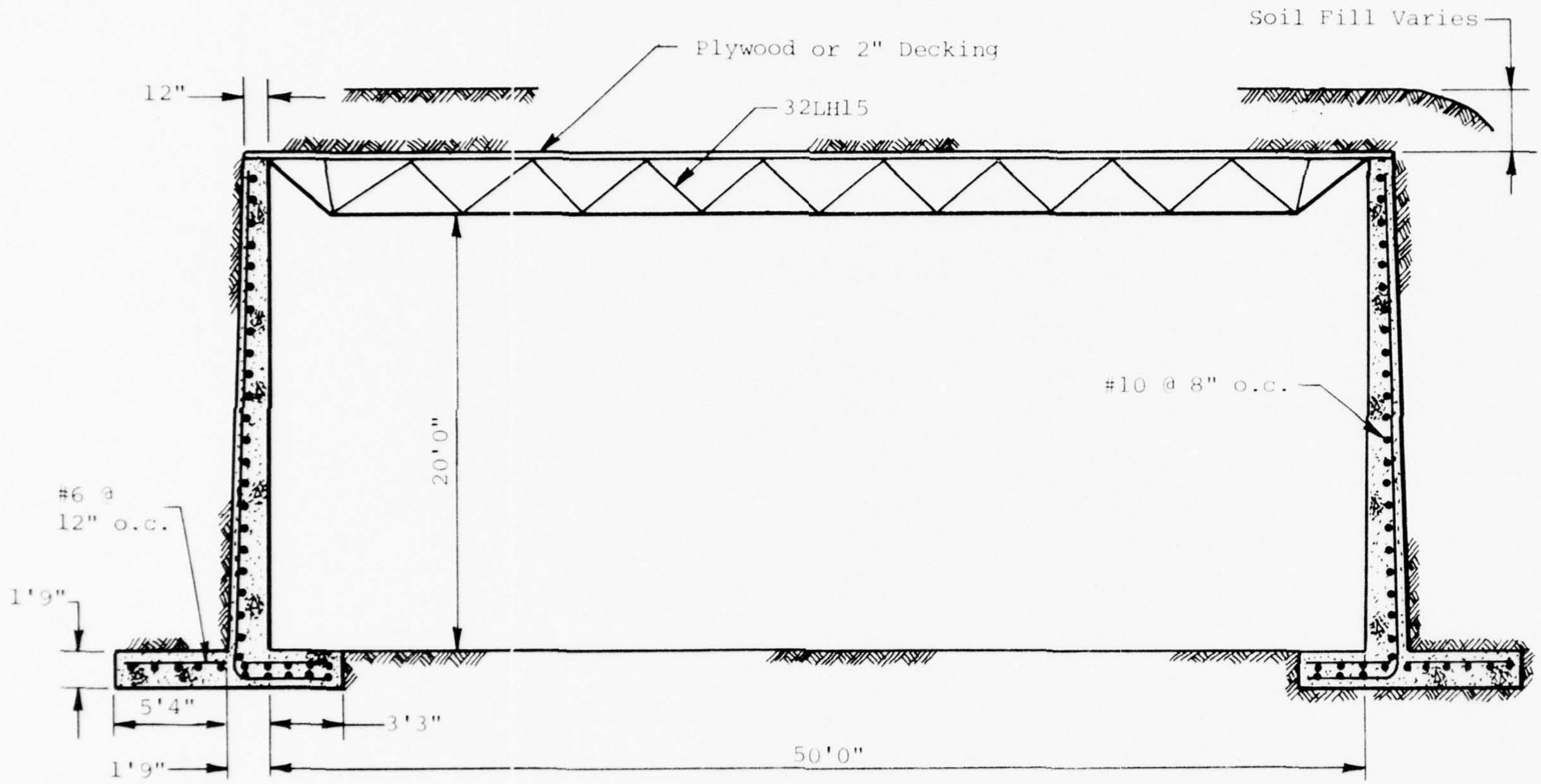


Figure 29. Long-Span Steel Joists - 50-Foot Span

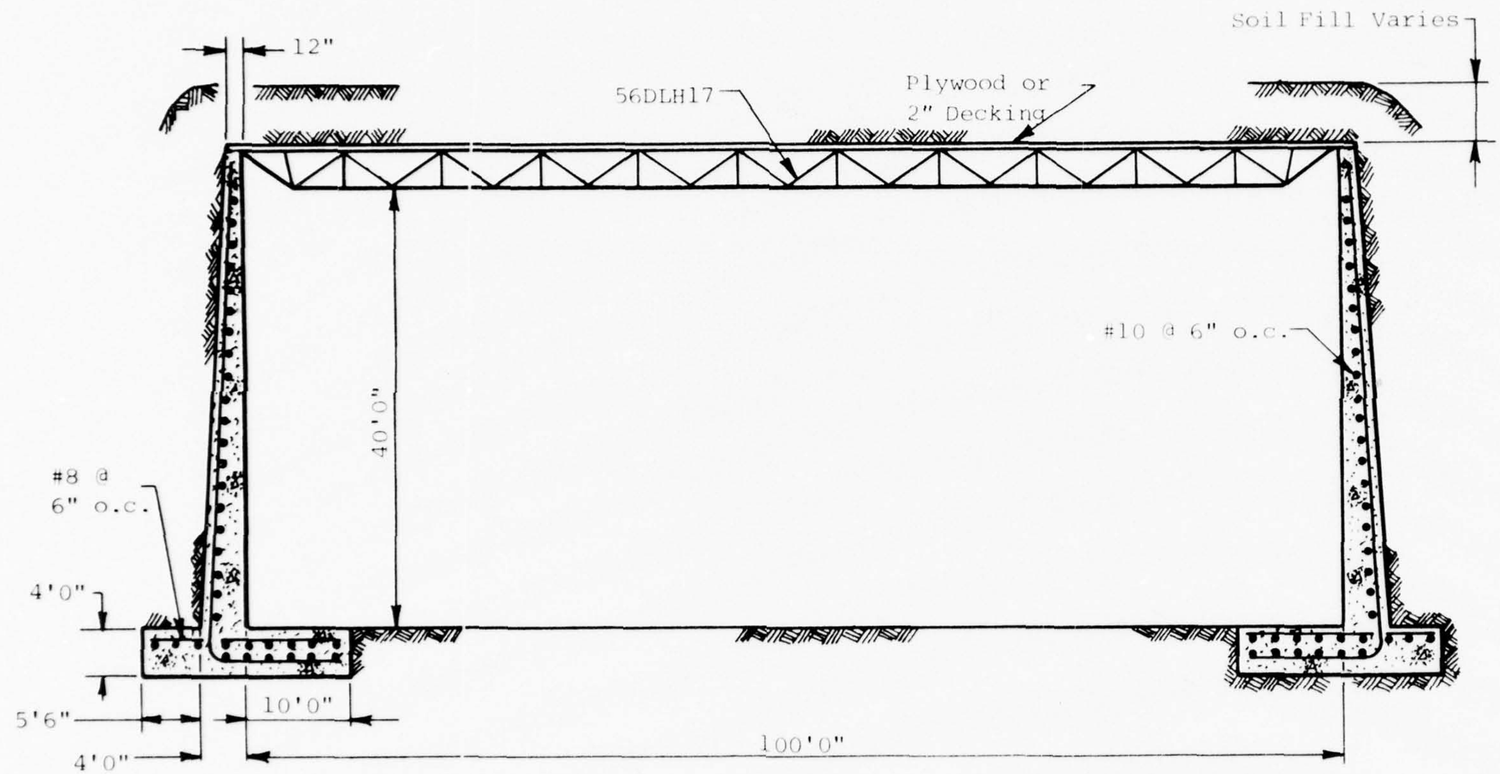


Figure 30. Long-Span Steel Joists - 100-Foot Span

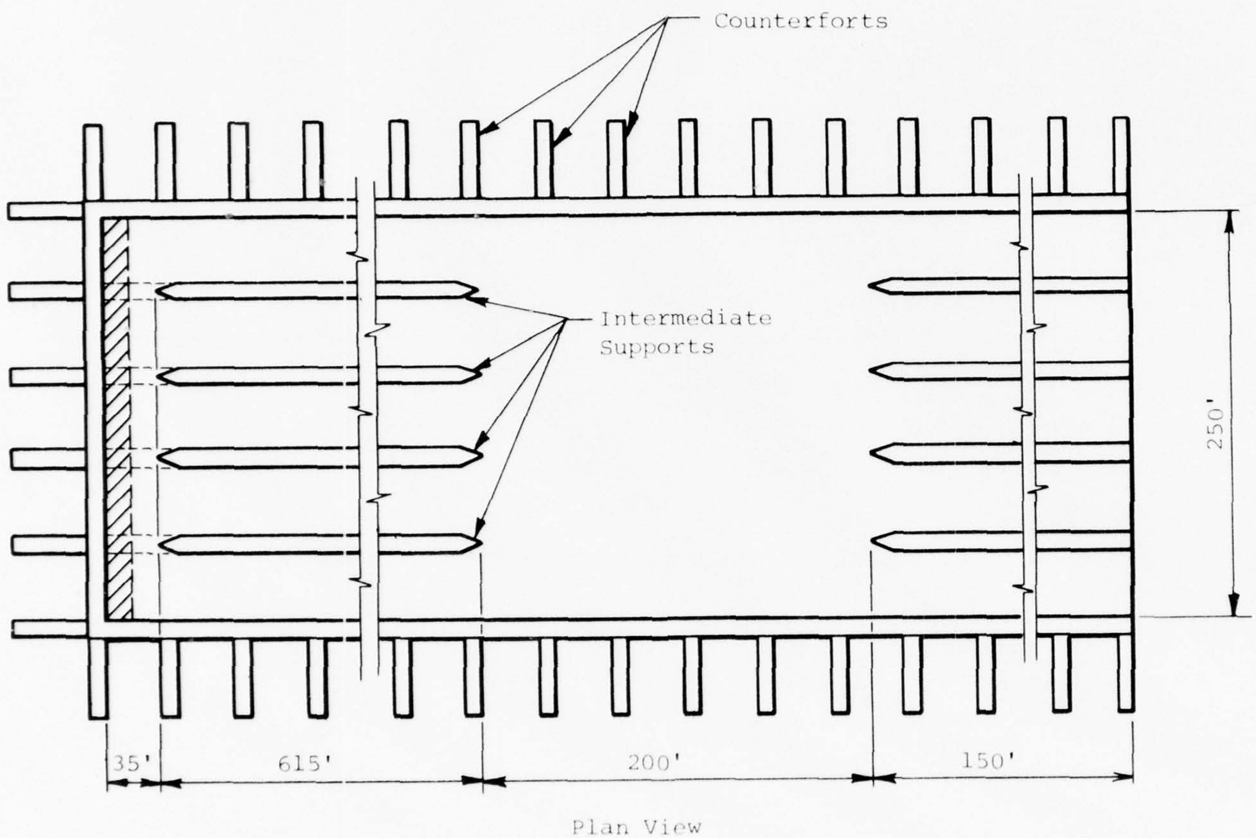
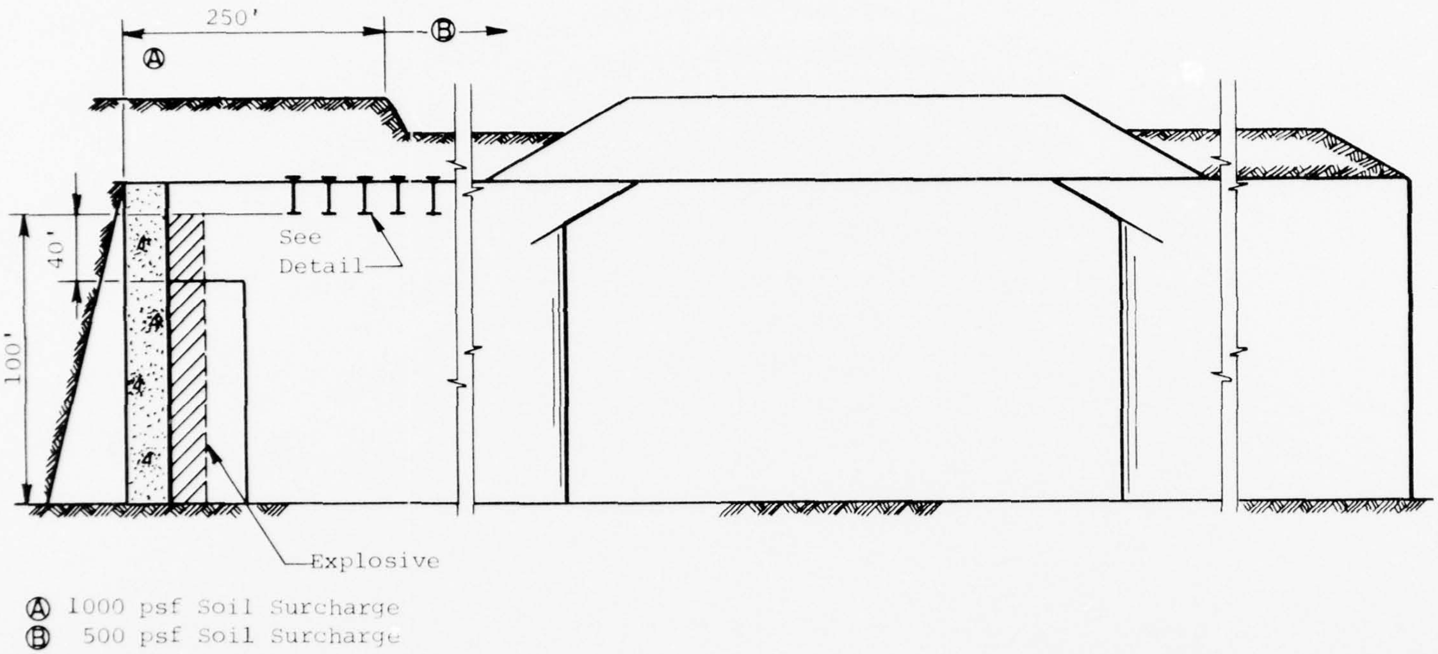
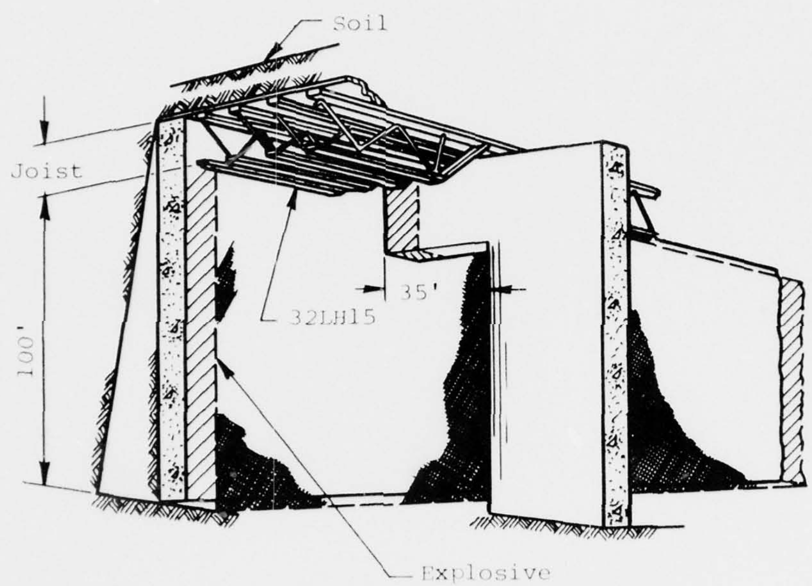


Figure 31a. Long-Span Steel Joists - 250-Foot Span



Longitudinal Section

Figure 31b. Long-Span Steel Joists - 250-Foot Span (Continued)



Explosive Section Detail

Figure 31c. Long-Span Steel Joists - 250-Foot Span (Concluded)

materials are also useable. The joists are spaced closely together resulting in a low decking material strength requirement.

The underside of the trusses supporting the overburden creates a very rough surface which could cause turbulence within the test cavity. The effect of this turbulence may be felt on portions of the test structure.

The material quantities and cost estimates per linear foot are

50-Foot Span

Trusses	1,110 lbs	@ $	1 =	$ 1,110	
Decking	52 sf	@	1 =	52	
20 Ft Retaining Wall	3.4 cy	@	400 =	1,360	
				$ 2,522	plf

100-Foot Span

Trusses	3,900 lbs	@ $	1 =	$ 3,900	
Decking	105 sf	@	1 =	105	
40 Ft Retaining Wall	13.2 cy	@	400 =	5,280	
				$ 9,285	plf

The cost of the 250-foot span has not been included because the scheme is clearly economically unfeasible.

6. COMPOSITE DECK SYSTEM

This concept utilizes a reinforced concrete deck that acts monolithically with structural steel shapes that support the deck. The preliminary dimensions and member sizes have been determined for a 50-foot span (fig. 32) and for a 100-foot span (fig. 33). The concrete deck is the same for both spans.

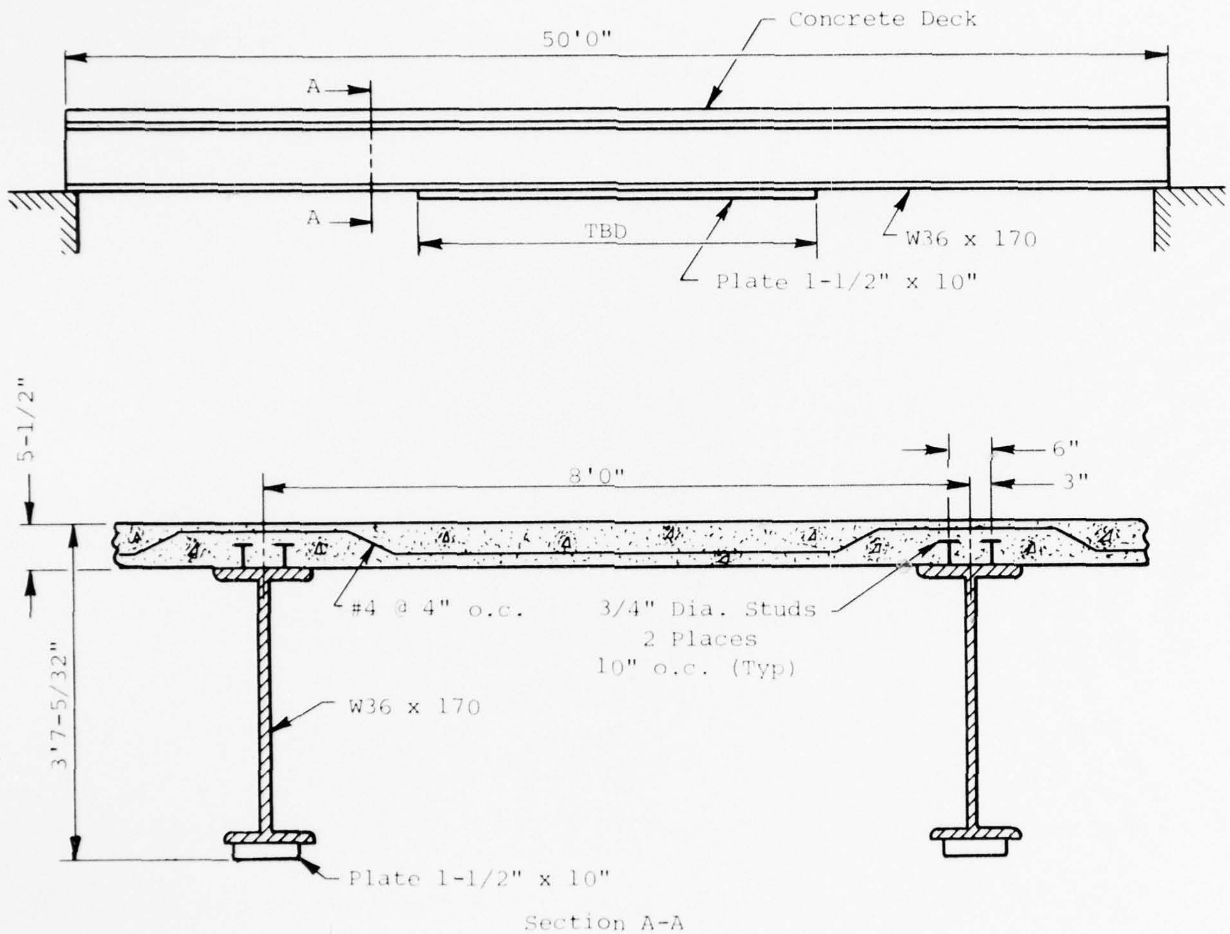


Figure 32. Composite Deck Concept - 50-Foot Span

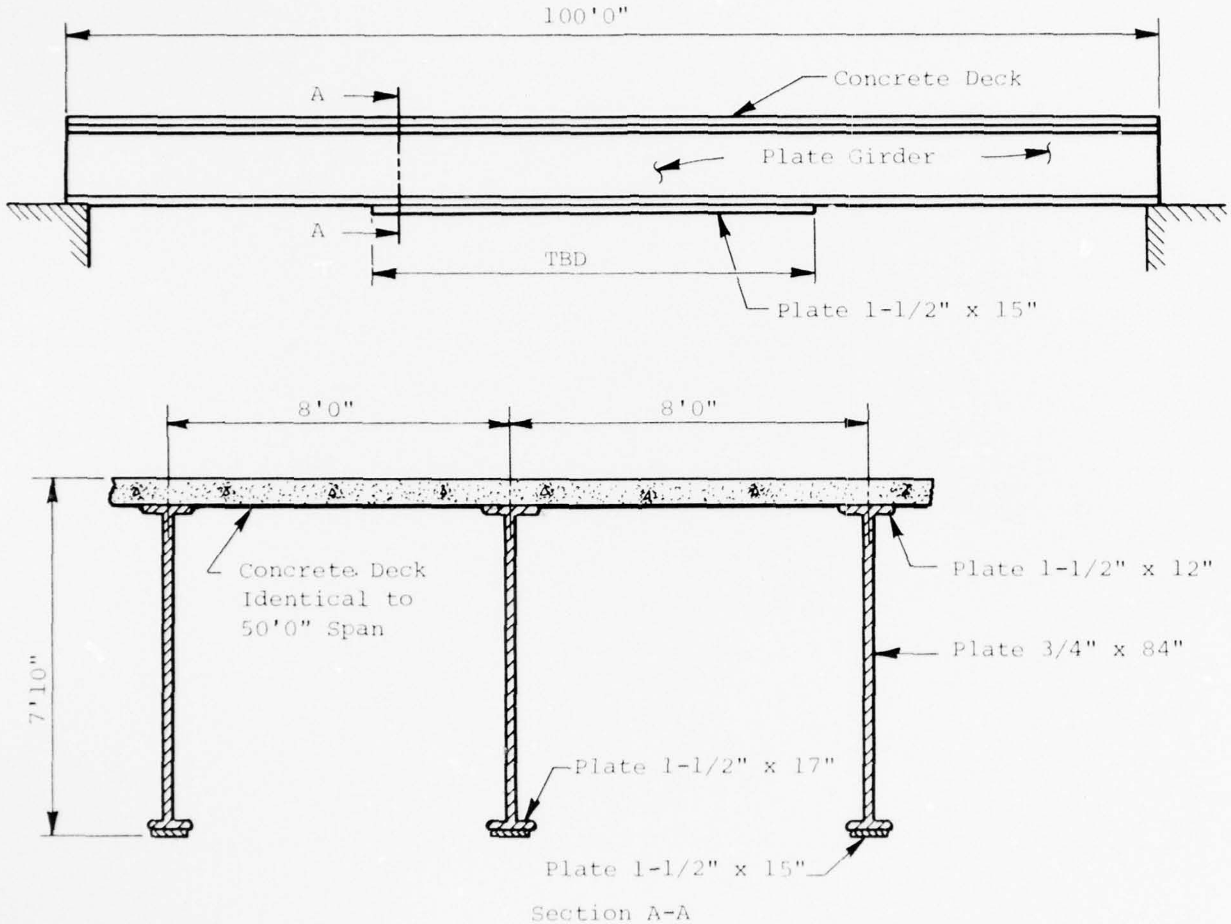


Figure 33. Composite Deck Concept - 100-Foot Span

The foundations and construction techniques for this concept will be similar to those used for freeway bridge construction. The steel shapes protrude into the test cavity and may cause some turbulence within the test cavity. The effect of this turbulence might be felt on portions of the test structure. Cover sheets at the bottom flanges could prevent or reduce this turbulence at increased cost.

The material quantities and cost estimates per linear foot are

50-Foot Span

Concrete	0.849 cy	@ $400 =	$ 340	
Structural Steel	1,382 lbs	@ 2 =	2,764	
20 Ft Retaining Wall	3.4 cy	@ 400 =	1,360	
			$ 4,464	plf

100-Foot Span

Concrete	1.698 cy	@ $400 =	$ 680	
Structural Steel	5,482 lbs	@ 2 =	10,964	
40 Ft Retaining Wall	13.2 cy	@ 400 =	5,280	
			$16,924	plf

The cost of the 250-foot span has not been included because the concept is clearly economically unfeasible.

7. AIRCRAFT SHELTER ARCH

This concept employs metal liners such as used for TAB VEE aircraft shelters. The metal arch liner is a very flexible structure and may require stiffening. Lining the entire structure with concrete would be expensive. The proposed designs (figs. 34 and 35) use a partial concrete liner extending

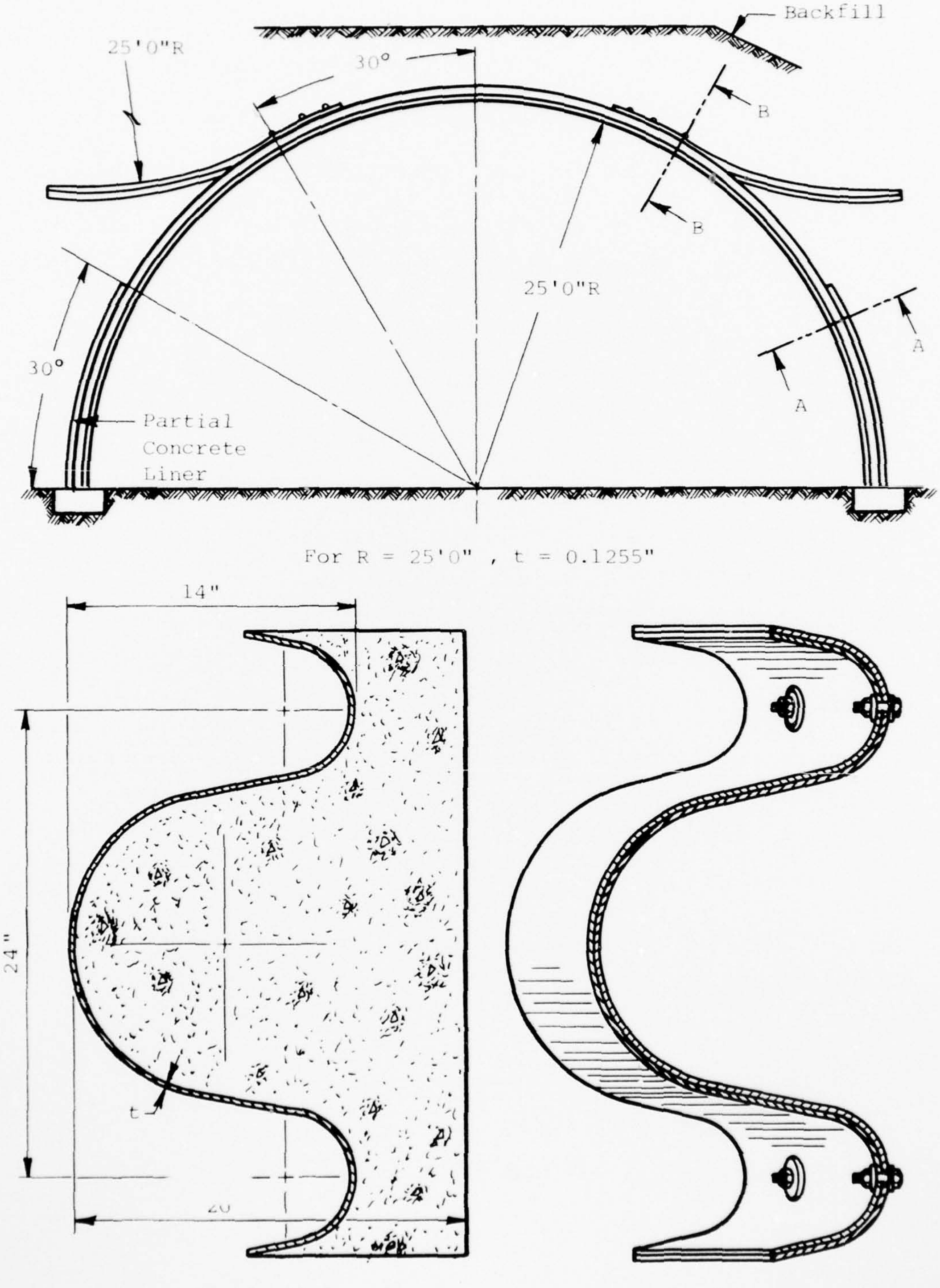


Figure 34. Aircraft Shelter Arch - 50-Foot Span

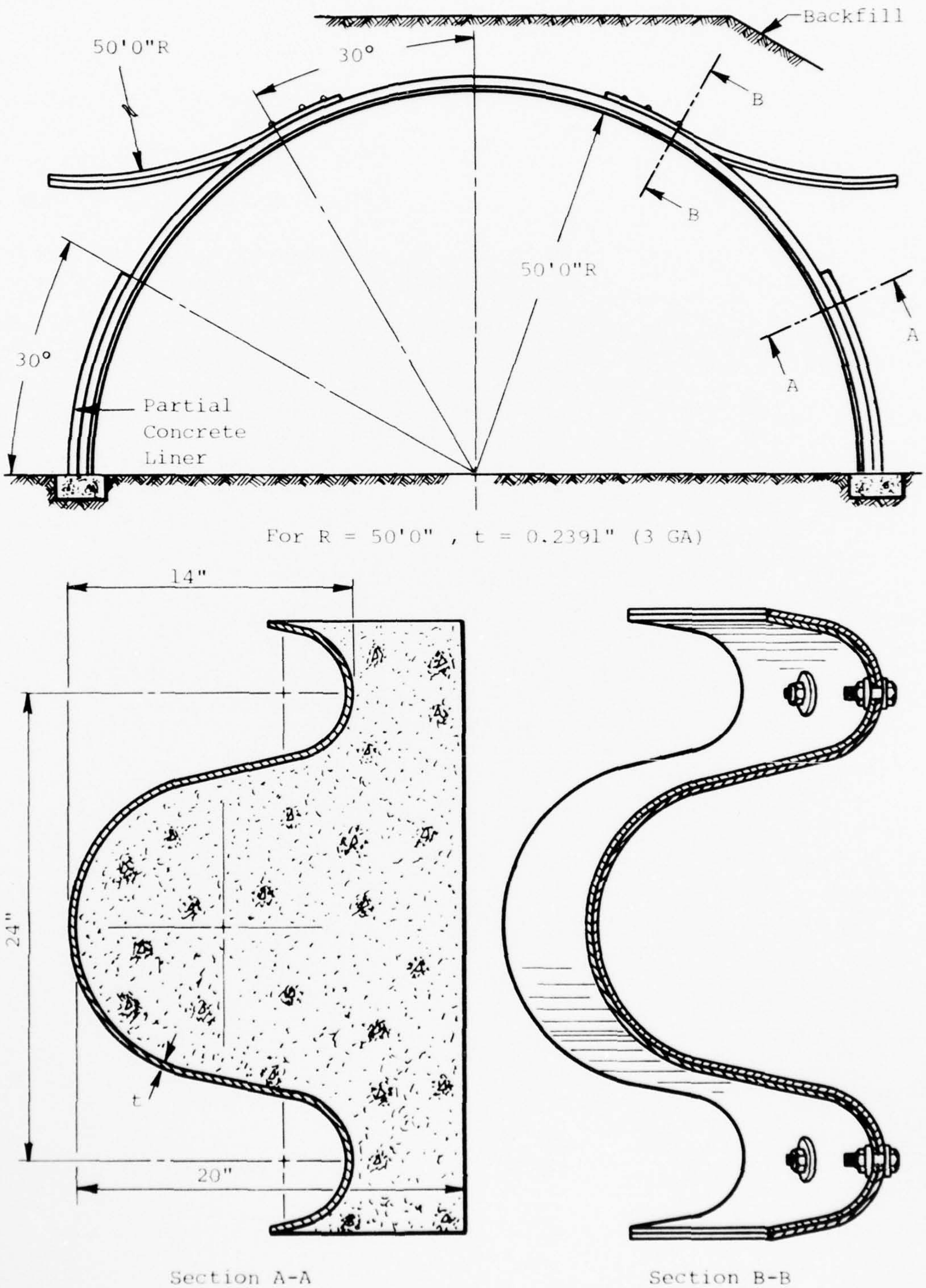


Figure 35. Aircraft Shelter Arch - 100-Foot Span

to the 30-degree line in the arch. To further strengthen the liner, a wing support is added at the 60-degree line in the arch. The wing, placed on the liner when the backfill reaches its elevation, will help restrain deflections so that minimum cover over the crown can be placed safely. When properly restrained, the arch can theoretically be covered until the metal yields.

The chief advantages of this concept are that the side wall and foundation requirements will be less than for any other concept. The main disadvantage is the care required during the fill operations. Another disadvantage is that the depth of the corrugations may adversely affect the shock wave simulation.

The material quantities and cost estimates per linear foot, excluding the arch footings, are

50-Foot Span

Arch Liner	678 lbs	@ $ 1	=	$ 678
Wings	200 lbs	@ 1	=	200
Concrete	1.5 cy	@ 400	=	600
				$1,478 plf

100-Foot Span

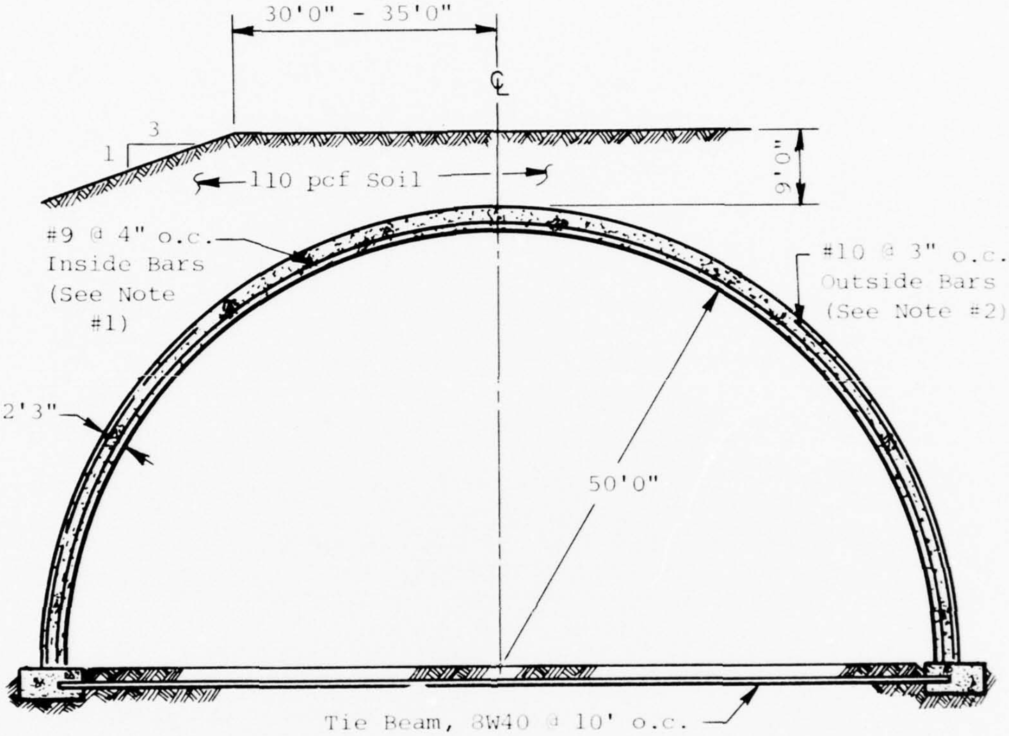
Arch Liner	2,580 lbs	@ $ 1	=	$2,580
Wings	800 lbs	@ 1	=	800
Concrete	2.9 cy	@ 400	=	1,160
				$4,540 plf

The aircraft shelter-type metal liners are not feasible for a 250-foot span.

8. CIRCULAR CONCRETE ARCH

A 100-foot span reinforced concrete cylindrical arch was investigated to provide a basis for comparison of the concept with others at the intermediate span. Earth cover provides the required depth of surcharge. The concept does not require side retaining walls or decking to support the surcharge, since the arch performs both of these functions. Tie beams are required to provide horizontal reactions at the springline of the arch. The relatively thin arch wall should not result in as much debris for posttest cleanup as some of the concrete sections proposed in other concepts. It may also be possible to modify the arch shell to induce a break at the crown under internal blast loading and help minimize the fallback of debris onto the test bed.

A preliminary design of the 100-foot span arch of uniform thickness is shown in figure 36. Although this design uses a uniform thickness arch shell, other concepts may prove more attractive. One alternative is a rib-stiffened circular arch. This type of arch would have a thinner wall than the one shown in figure 36, but increased costs due to more complex forming and reinforcing details would at least partially offset savings due to reduced material quantities. The rib-stiffened arch also poses more difficult foundation design problems than does the uniform thickness arch. Cost estimates were not prepared for the rib-stiffened arch.



Note 1: Inside Bars Continuous Around Arch

Note 2: Outside Bars Cut Off as Shown

Figure 36. Circular Concrete Arch - 100-Foot Span

The material quantities and cost estimate per linear foot, excluding the arch footings, are

100-Foot Span

Concrete	13.1 cy	@ $400 =	$5,240	
Steel Tie Beams	400 lbs	@ 2 =	800	
			$6,040	plf

9. COST SUMMARY

The cost summary for the concepts described in this section is presented in table 2. Costs are shown in dollars per linear foot of test bed.

Table 2

COST OF PROPOSED DABS CONCEPTS

Concept	Span		
	50-Foot ($ plf)	100-Foot ($ plf)	250-Foot ($ plf)
Parabolic Arch w/ Suspended Platform	2,194	7,600	40,834
Cable Suspension System	N/A	14,240	74,000
Concrete Box Girder	2,000	8,000	50,480
Long Span Steel Joist	2,522	9,285	N/A
Composite Deck System	4,464	16,924	N/A
Aircraft Shelter Arch	1,478	4,540	N/A
Circular Concrete Arch	N/A	6,040	N/A

SECTION V

EVALUATION OF SELECTED DABS CONCEPTS

1. INTRODUCTION

Structures capable of providing clear spans of from 50 to 250 feet were described in the previous section. The concepts included

- Parabolic Arch with Suspended Platform.
- Cable Suspension System.
- Concrete Box Girder.
- Long-Span Steel Joist.
- Composite Deck System.
- Aircraft Shelter Arch.
- Circular Concrete Arch.

The circular concrete arch and box girder were selected by the Air Force for continued development. Long-span joists were considered as alternates for box girders where spans and load combinations were satisfied by commercially available long-span steel joist designs.

The doubly corrugated steel aircraft shelter arches were also investigated for one-half scale DABS facilities. While these structures appear attractive from the viewpoint of cost and construction time, their load carrying capacity is difficult to predict analytically. Design data for a 75-foot semicircular arch was obtained from the Wonder Building Division of Modular Technology Corporation. The data provided were

for material thicknesses up to 0.11 inch and reportedly represent the current maximum capability of the manufacturer. Analysis indicates that this arch would not safely carry the 500 psf surcharge load; however, it is also recognized that these types of structures will often withstand loads higher than those indicated by conventional methods of analysis. The steel arches have not been completely ruled out as a concept, but they must be considered an option that must be examined experimentally to determine their range of usefulness.

Structural designs were based on the Air Force Weapons Laboratory criteria presented in table 3 and figures 37 and 38. In addition to these criteria, it was decided that intermediate supports would be acceptable in locations other than the test section, if they provided a reduction in facility costs. Intermediate supports in the test section were considered acceptable only for the full-scale facility and only if they did not impose large concentrated dead loads on the test items. It was concluded that intermediate supports should be continuous walls to minimize the possibility of debris impacting upon the test items.

Cost estimates include all structural components, foundations, endwalls in the driver sections, placement of surcharge, and site restoration. Site restoration consists of burying debris at least 2 feet below ground surface and restoration of the area to the general pretest contours. Costs

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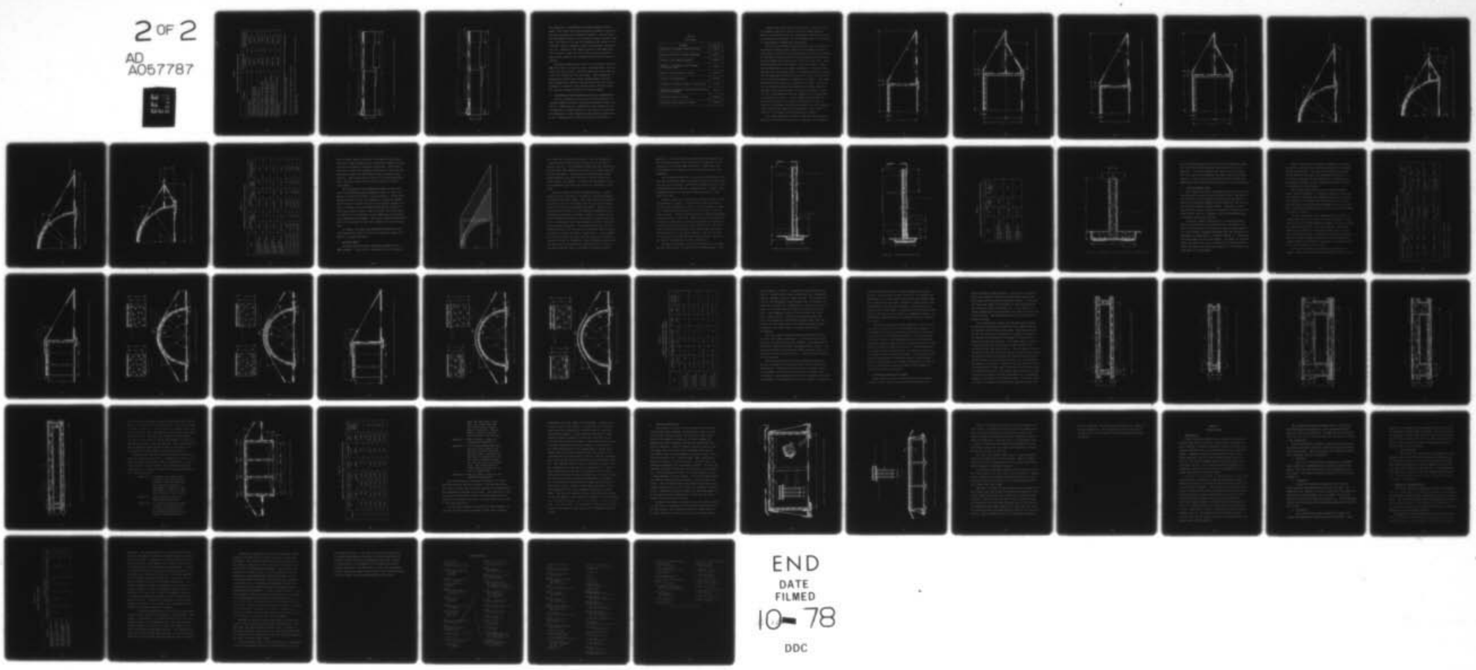

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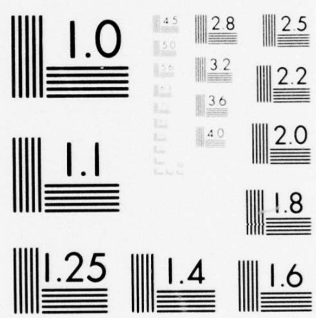

MICROCOPY RESOLUTION TEST CHART
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Table 3

DESIGN CRITERIA FOR DABS FACILITIES

Criteria	Full-Scale Test Facility	One-Half Scale Test Facility
Test Bed Width	200 ft	100 ft[1]
Overall Length of Facility (including driver section)	800 ft	400 ft
Length of Driver Section	20 ft	10 ft
Maximum Clearance over Test Bed	100 ft[2]	50 ft[2]
Weight of Surcharge at Test Section (includes weight of structure)	1000 psf	500 psf
Weight of Surcharge at Driver Section (includes weight of structure)	2000 psf	1000 psf
Bearing Capacity of Soil for Foundation Design	4400 psf[3]	4400 psf[3]

[1]Span of 70 to 80 feet may be acceptable if it permits use of commercially available corrugated metal arches.

[2]For arch type structures measured at midspan.

[3]Deviation from this criterion was necessary in order to obtain realistic foundation designs.

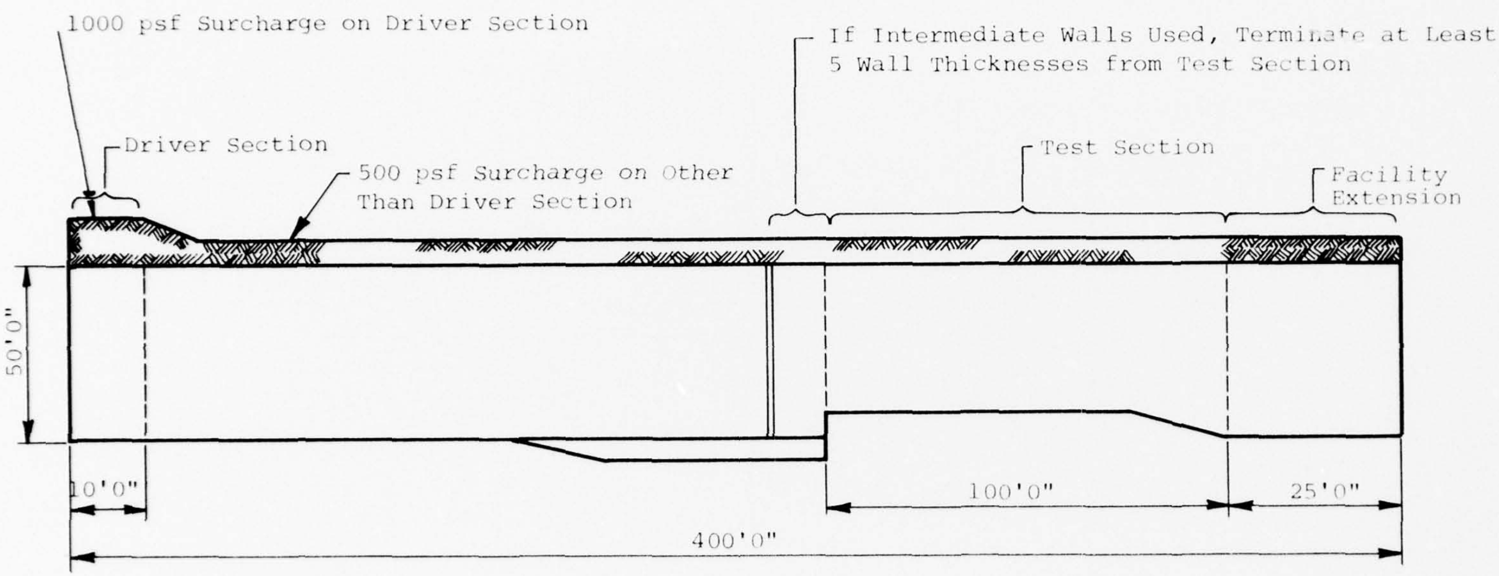

Figure 37. Longitudinal Section Through One-Half Scale DABS Facility

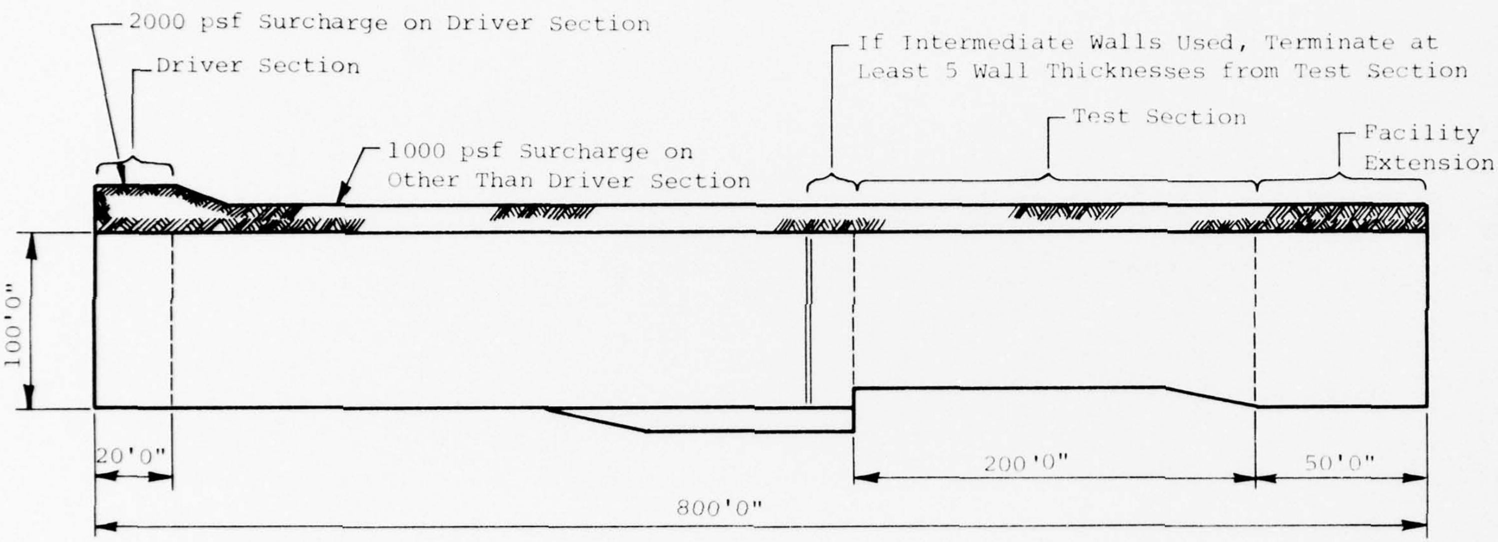

Figure 38. Longitudinal Section Through Full-Scale DABS Facility

are estimated for aboveground and optimum depth of burial cases. Unit costs for various elements included in the facility cost estimates are presented in table 4. The unit cost for reinforced concrete is an estimated average cost per cubic yard. The actual unit cost will depend on the amount of reinforcing, type of formwork, height of placement, strength of mix, etc. Because of the unusual nature of the DABS structures, material quantities are a more realistic measure of the relative costs of the concepts than the dollar totals indicated.

Preliminary designs and costs for two general classes of DABS facilities (a full-scale and a one-half scale) are presented for each concept. Multiple spans and steel truss options are included for the box girder. Also considered is aboveground construction versus optimum depth of burial. This consideration affects only the amount and cost of the earthwork to be done in the construction of the facilities. The various concepts presented are then evaluated.

2. ABOVEGROUND VERSUS BALANCED CUT AND FILL CONSTRUCTION

All the proposed concepts involve large quantities of earthwork which represent a significant portion of the total cost of a DABS facility. Depending on soil conditions and availability of borrow material at the test site, there may be advantages to either completely aboveground construction or to a combination of balanced excavation and backfill.

Table 4

UNIT COSTS

Element	Cost ($)
Excavate by Scraper (1500-foot haul)	1.25/cy
Direct Excavation by Other Machines	2.00/cy
Backfill with Small Equipment	2.00/cy
Backfill and Compact with Scrapers (1500-foot haul)	1.00/cy
Borrow (load and haul 2 miles)	2.50/cy
Grading and Compaction	0.50/sy
Disposal of Concrete Debris (5-mile haul)	3.50/cy
Structural Concrete, Including Reinforcing Steel and Formwork	300/cy
Structural Steel In Place	1500/ton
Long Span Steel Joists In Place	750/ton

Ground water, type and bearing capacity of soils at various depths, costs and length of haul for borrow material, amount of precipitation during construction, and relative cost of excavation and backfill are all considerations in the choice of aboveground or belowground construction.

The relative earthwork quantities and costs of above and belowground locations were studied for rectangular and arch cross sections with 100 and 200-foot spans. Figures 39 through 46 represent transverse cross sections through the rectangular and arch shapes for one-half and full-scale facilities. Final berm dimensions may vary somewhat from those shown in these figures depending on the actual dimensions of structural members and the side slopes chosen. Earthwork quantities (including the earthwork for berms at the driver end of the facility) for the various concepts are summarized in table 5. The downstream end of the facility is assumed to be open. Excavation for the aboveground case was assumed to be accomplished by equipment other than scrapers at a cost of $2.00/cy. Excavation for the belowground case was assumed to be accomplished by scrapers at $1.25/cy. It was assumed that one-half the backfill quantities would have to be placed at a cost of $2.00/cy by equipment smaller than scrapers to avoid imposing heavy concentrated loads on the structures. The remaining one-half of the backfill was estimated at $1.00/cy.

The costs summarized in table 5 show only slight advantages for the balanced cut and fill condition for rectangular

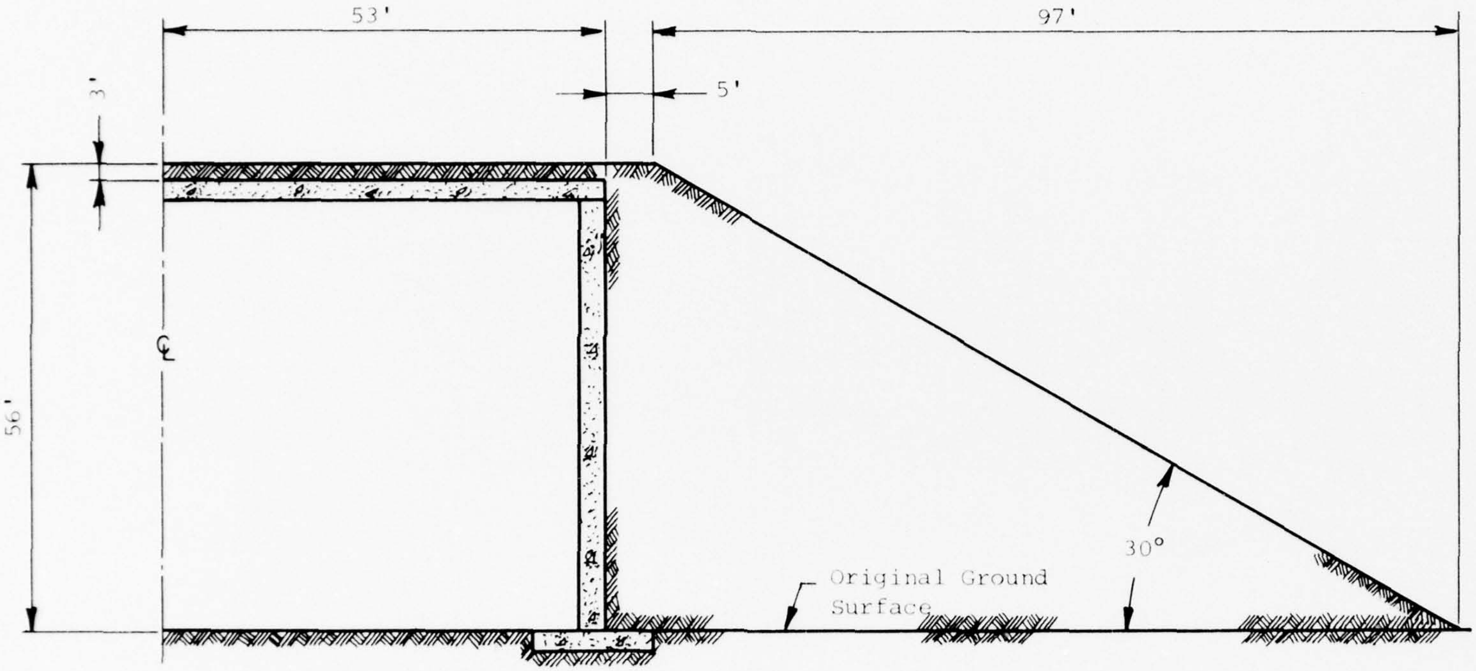

Figure 39. Aboveground Condition for 100-Foot Span Rectangular Section

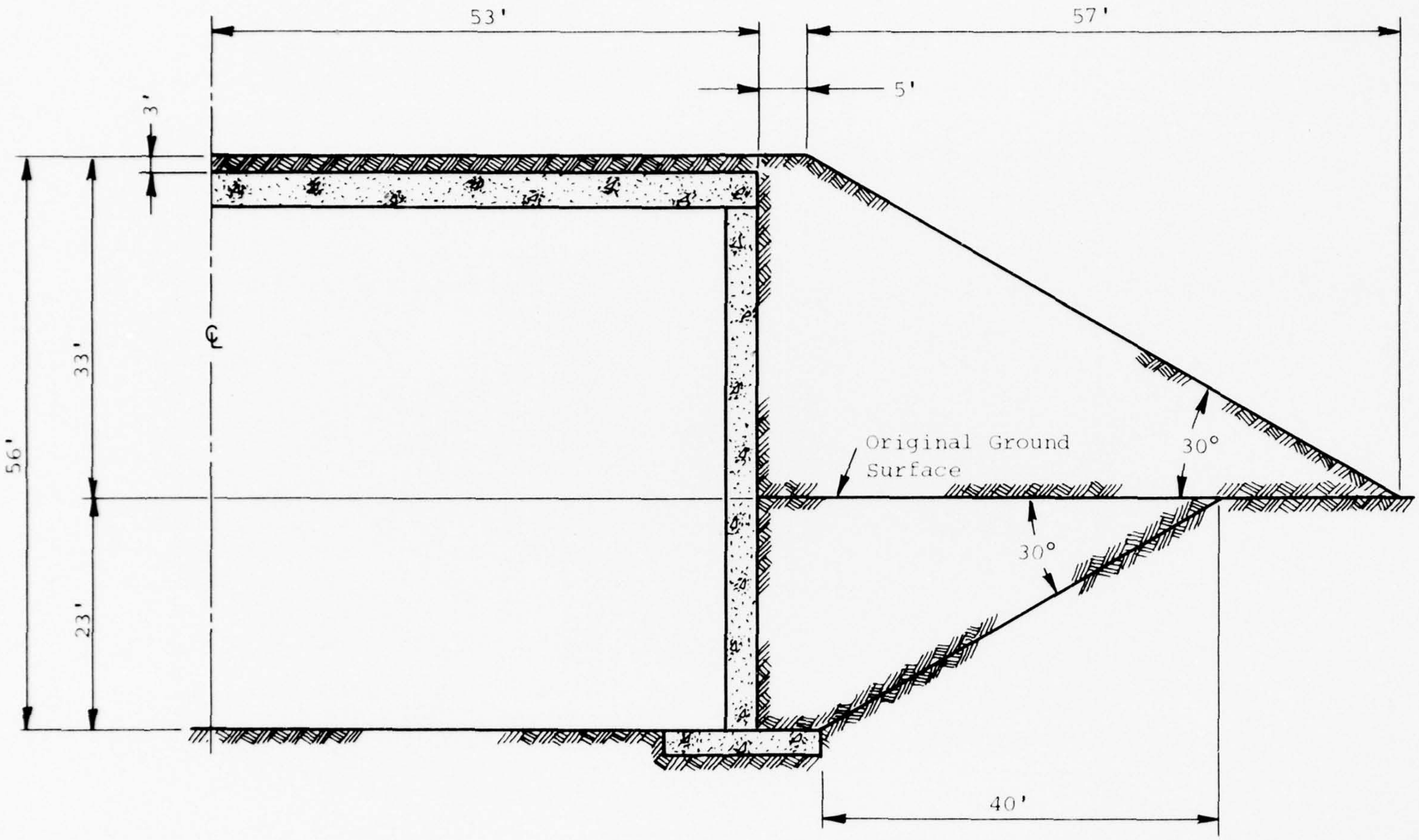

Figure 40. Balanced Cut and Backfill Condition for 100-Foot Span Rectangular Structure

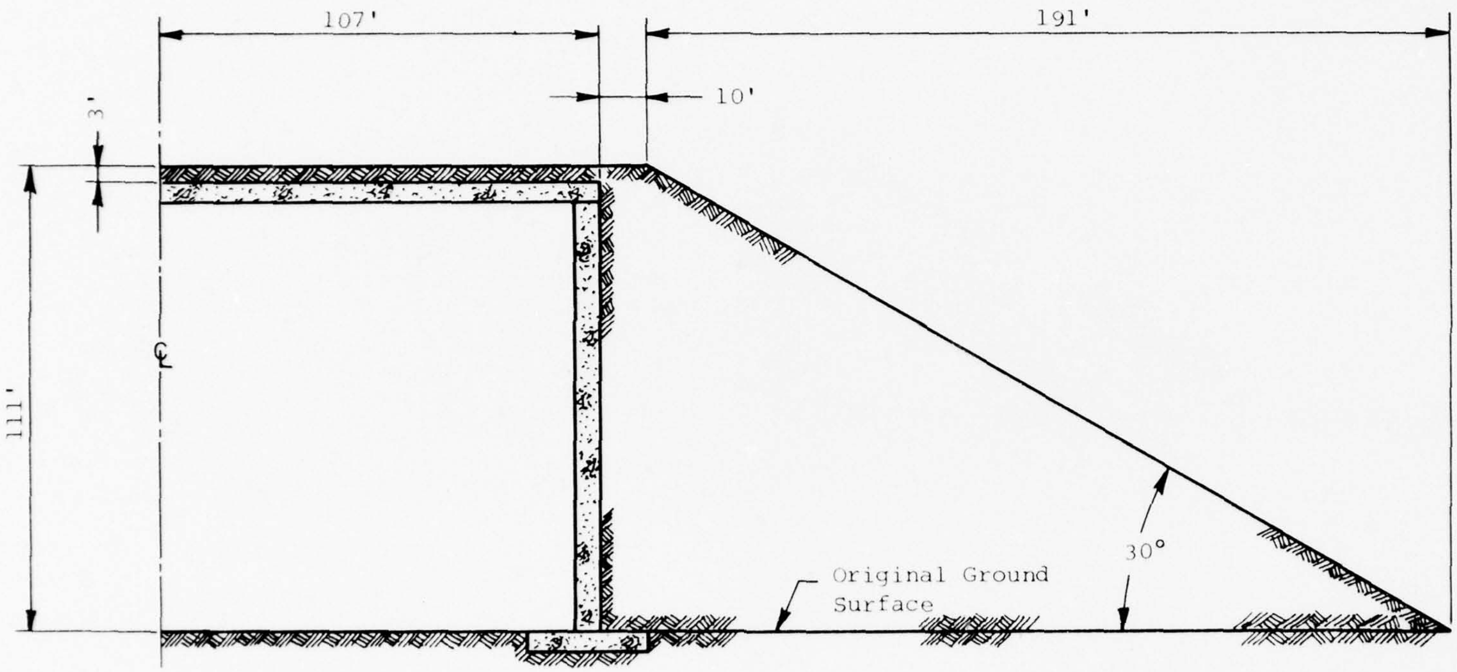

Figure 41. Aboveground Condition for 200-Foot Span Rectangular Section

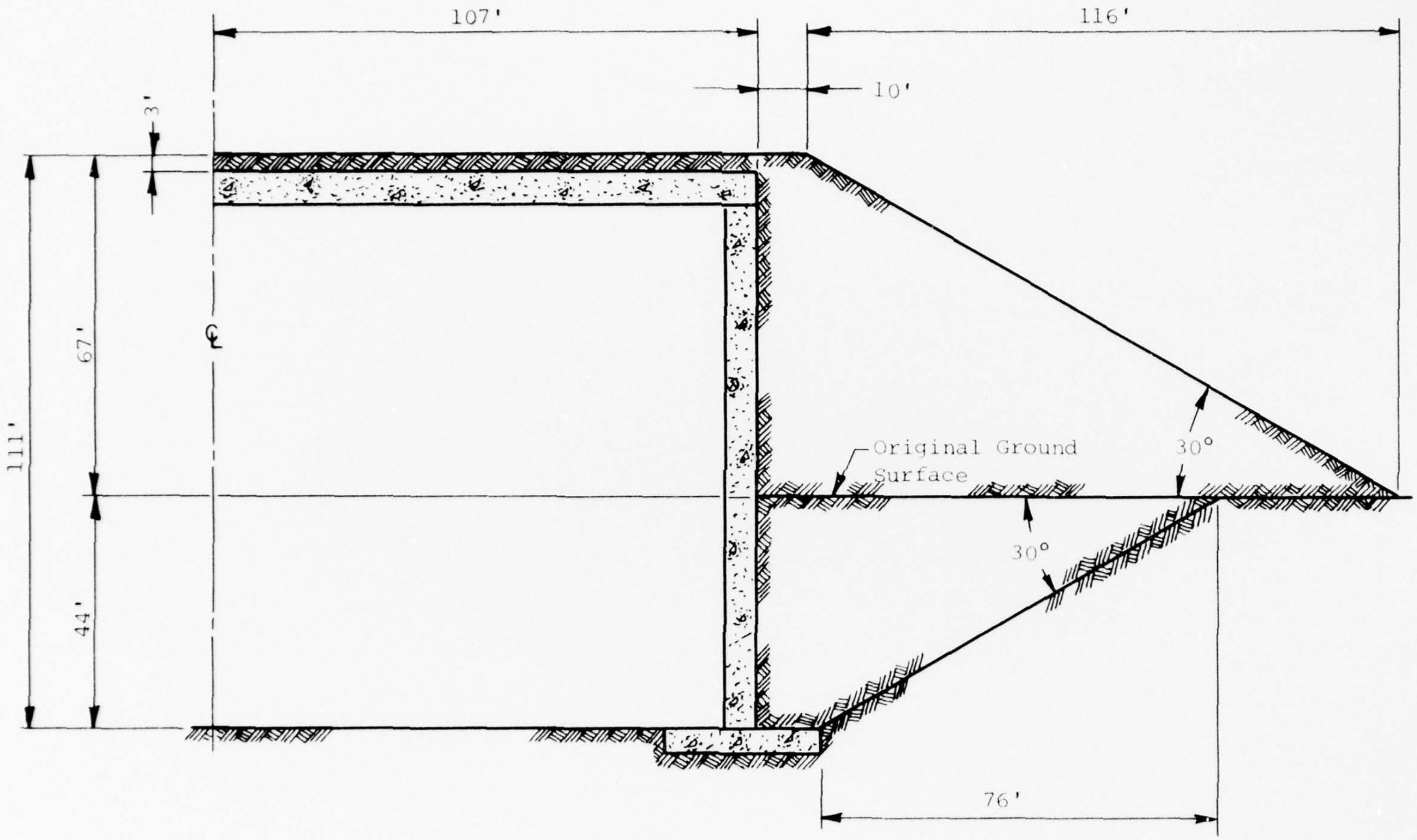

Figure 42. Balanced Cut and Backfill Condition for 200-Foot Span Rectangular Cross Section

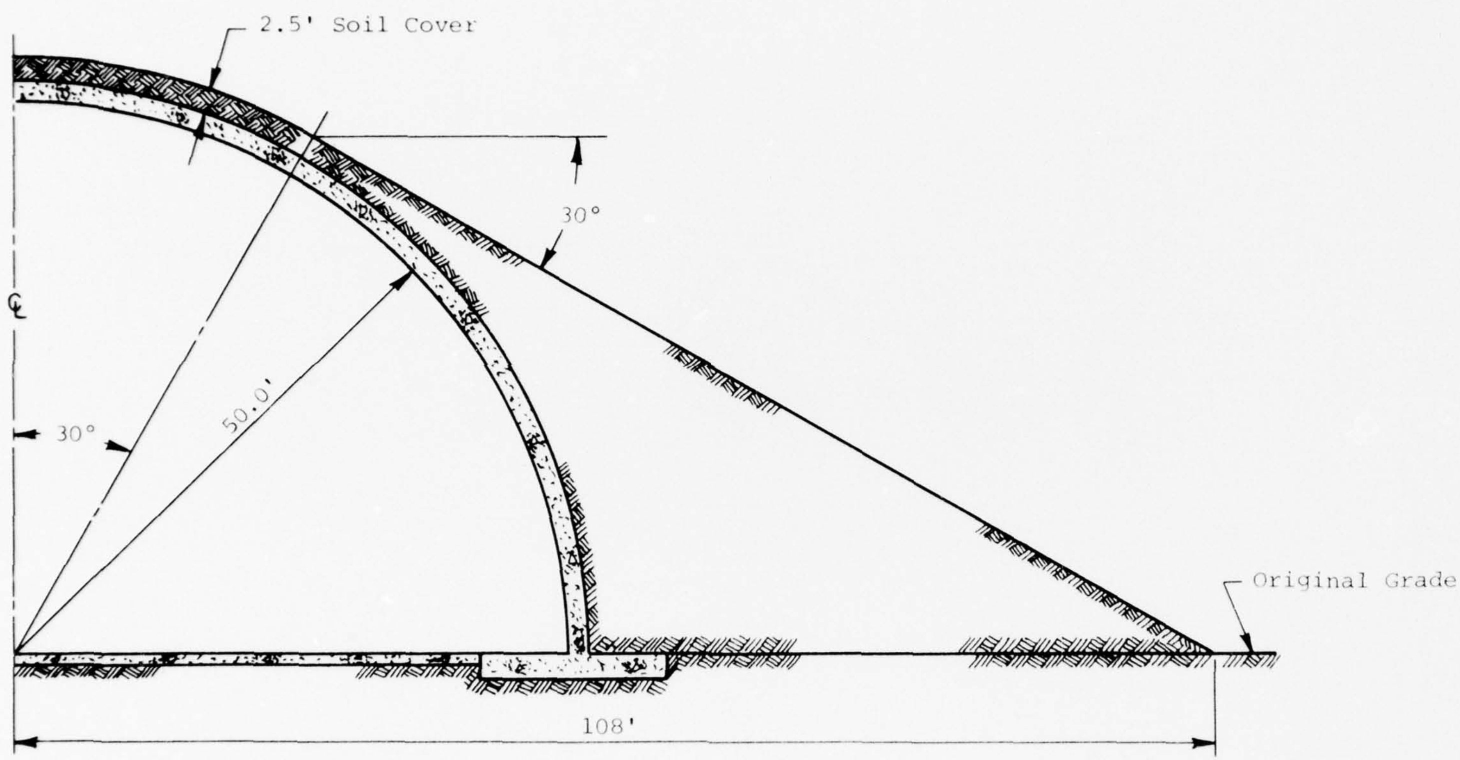

Figure 43. Aboveground Condition for 100-Foot Span Arch

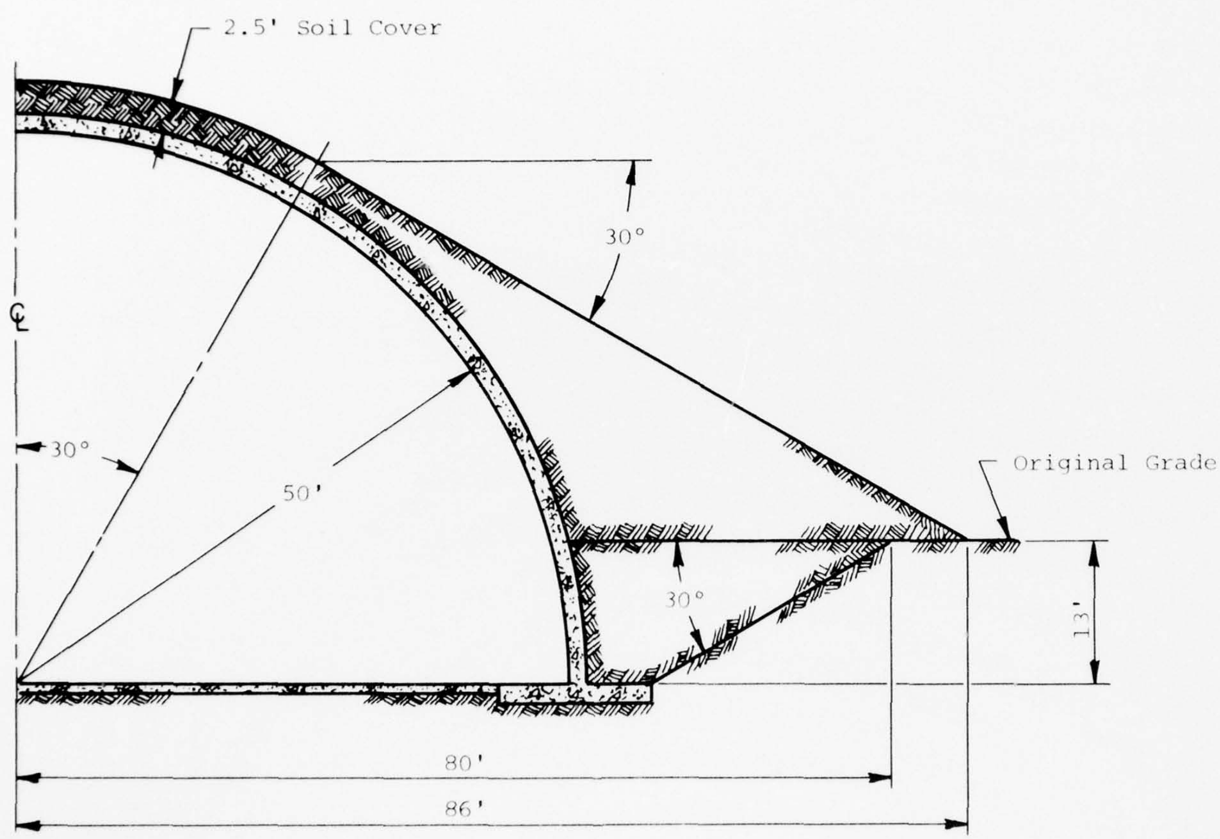

Figure 44. Balanced Cut and Fill Condition for 100-Foot Span Arch

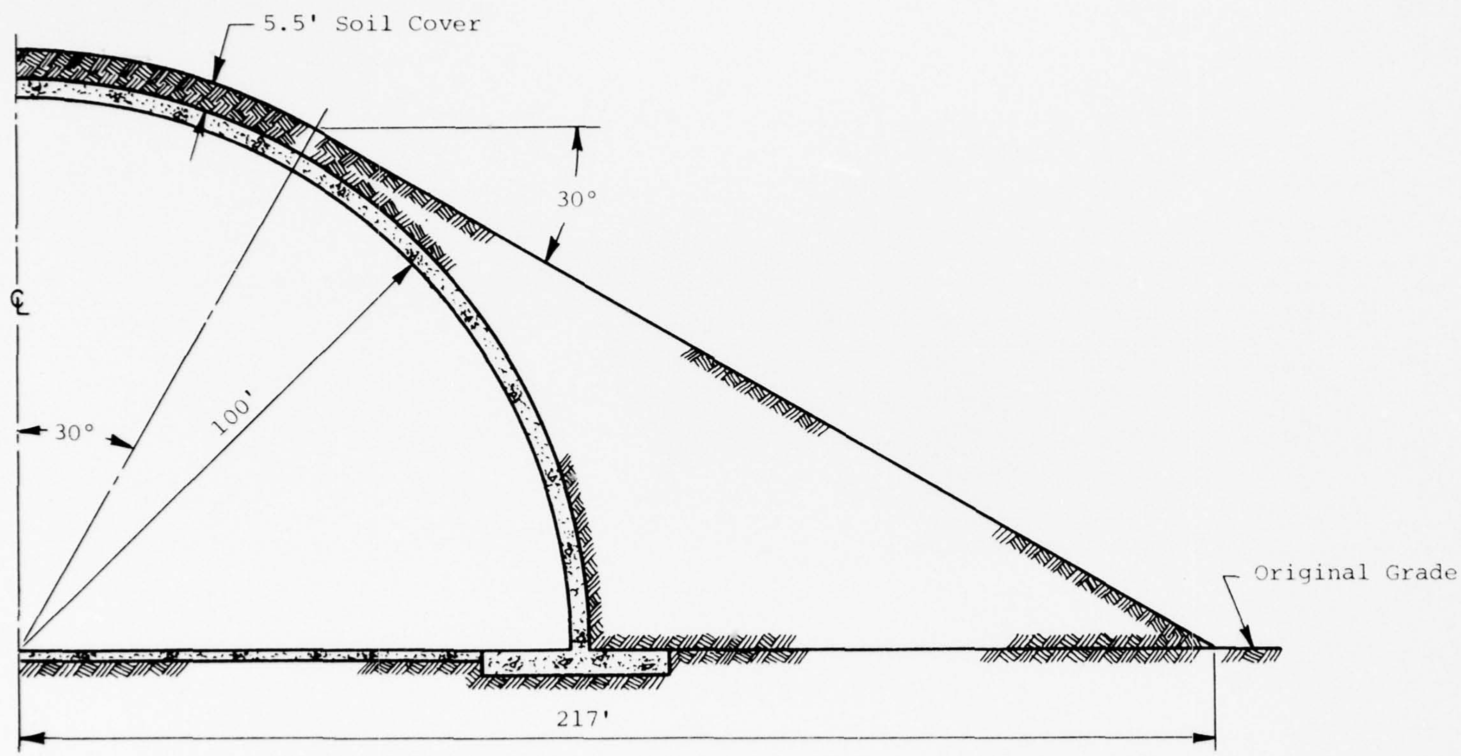

Figure 45. Aboveground Condition for 200-Foot Span Arch

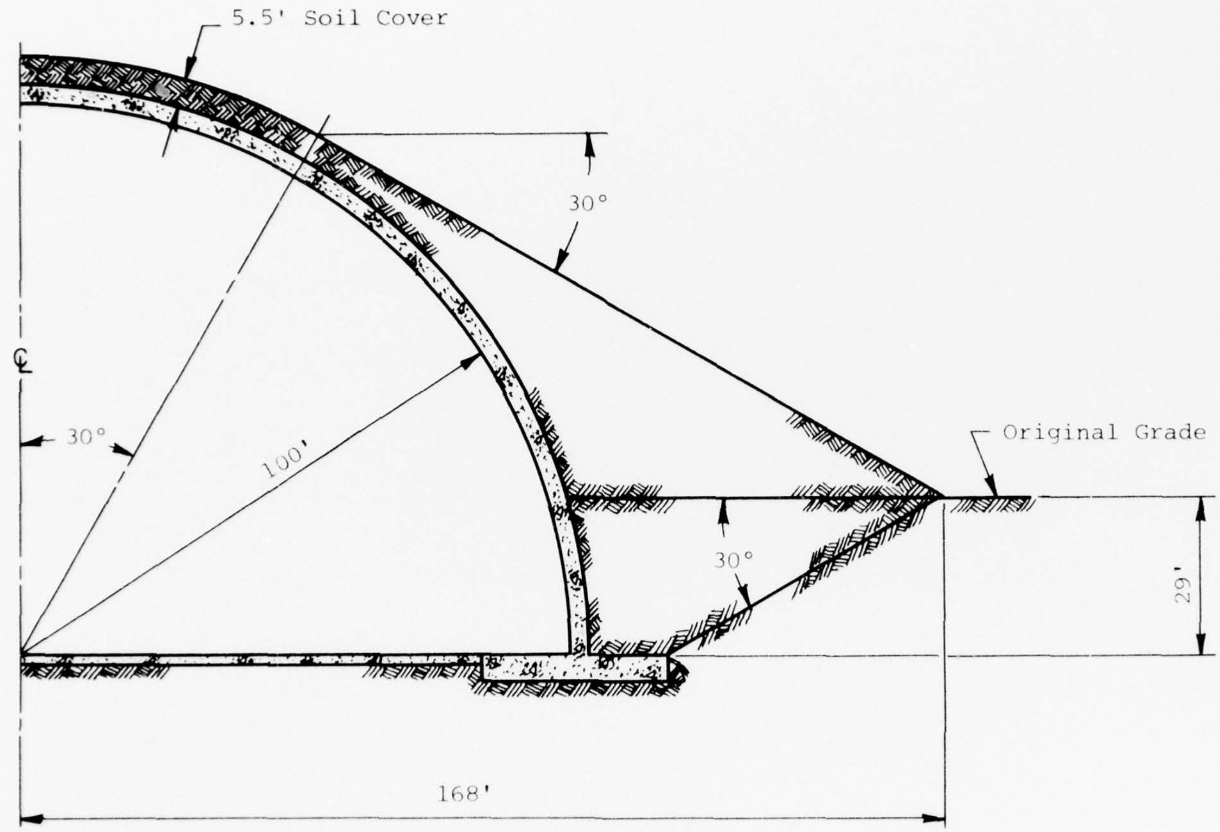

Figure 46. Balanced Cut and Fill Condition for 200-Foot Span Arch

Table 5

EARTHWORK QUANTITIES FOR VARIOUS CONCEPTS

Item	Excavation		Backfill		Total Cost ($000)	Cost per Foot of Test Section ($)
	cy	Cost ($000)	cy	Cost ($000)		
A/G 100-Foot Rectangular Test Section	1,100	2	121,100	182	184	460
B/G 100-Foot Rectangular Test Section	62,200	78	62,200	93	171	428
A/G 200-Foot Rectangular Test Section	11,300	23	1,035,800	1,554	1,577	1,970
B/G 200-Foot Rectangular Test Section	547,700	685	547,700	822	1,507	1,883
A/G 100-Foot Arch	1,000	2	55,600	83	85	213
B/G 100-Foot Arch	40,700	51	40,700	61	112	280
A/G 200-Foot Arch	9,800	20	454,900	682	702	877
B/G 200-Foot Arch	293,100	366	293,100	440	806	1,008

test sections, whereas the costs are higher for the arches. Obviously, slight changes in unit costs could change the overall costs and present a different conclusion. One definite conclusion is that the earthwork quantities for the arch test sections are much less than those for the rectangular test sections. This may be seen in figure 47 for the aboveground case. The much higher cost of the full-scale facility is also evident.

One advantage of the belowground concept is that post-test site restoration would probably be easier in that the debris can simply be covered in the excavation. In the aboveground condition, it will be necessary to remove the debris from the site to restore the site to the general pretest contours. Placement of arch foundations below the ground surface would provide greater lateral restraint and could result in reduction, or elimination, of the quantities of structural steel required for tie beams. This could offset the additional cost of the balanced cut and fill condition for the arch concept.

In summary, the above and belowground locations are so nearly equal that choice of one or the other will have to be made on a site-by-site basis.

3. RETAINING WALLS

Retaining walls are major structural elements in all DABS concepts. In the arch, a retaining wall is required at

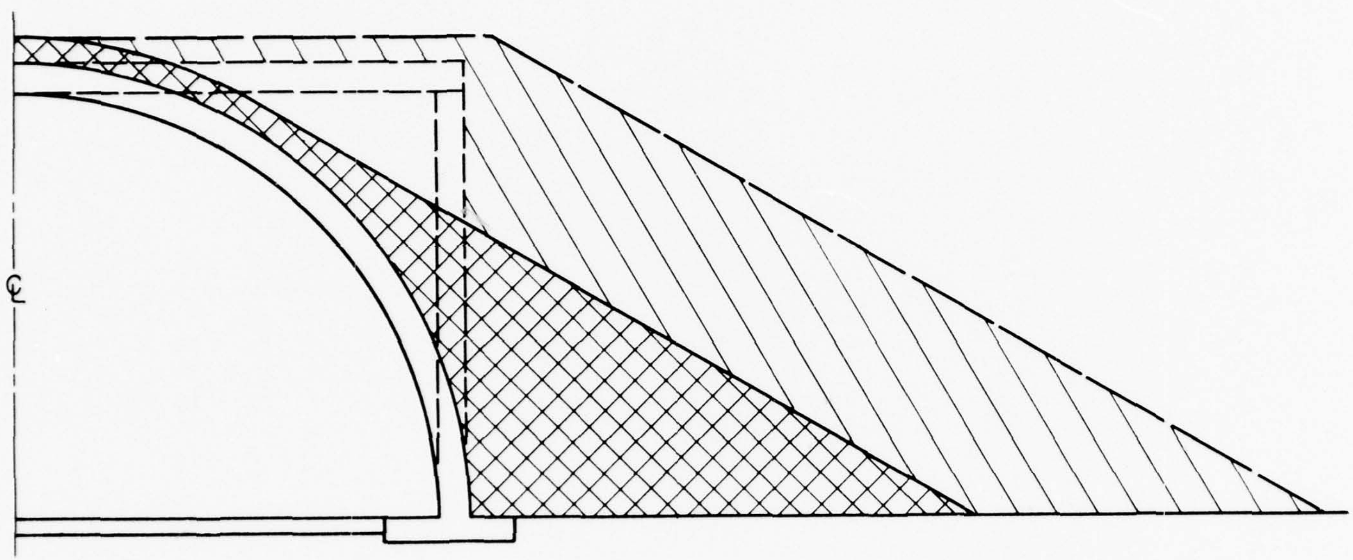

Figure 47. Comparision of Backfill Quantities for Rectangular and Arch Test Sections

the closed end of the driver section. For the rectangular cross sections, walls are also required along the sides to support the roof. For preliminary design and cost estimating, the end walls were assumed to be identical to those required for the side walls. The side walls must withstand lateral loads imposed by the earth cover and vertical loads transmitted by the roof system. In view of the magnitude of both load components, the side walls are major engineering structures.

There are several choices for the side walls. These include gravity, cantilever, counterfort, anchored bulkheads and propped cantilever types. The first three are freestanding and the last two are braced near the top edge and sometimes at the bottom and intermediate points. Except for the long-span steel joists, all roof systems considered for the rectangular structures are massive enough to provide the required horizontal reaction at the top of a propped cantilever wall. A counterfort wall would probably have to be used with a steel joist roof system. Preliminary designs for the side retaining walls of the one-half and full-scale DABS facilities are shown in figures 48 and 49, respectively. It was assumed that the soil behind the wall was a dry sand with an angle of internal friction of 30 degrees. Wall foundation designs were also based on these general soil characteristics in lieu of the 4400 pounds per square foot bearing capacity initially

specified. Very large horizontal loads are developed at the top and bottom edges of the retaining walls required for the one-half and full-scale DABS facilities. These loads must be considered in the design of the roof system and the wall foundation.

The foundations depicted in figures 48 and 49 cannot provide the required horizontal reaction and will have to be modified during final design. Two obvious possibilities are a key on the bottom of the footing or deeper burial. Another, less desirable, solution is the use of struts bracing the lower edge of the wall.

Material quantities and costs for the two sizes of walls are presented in table 6. The estimated costs are based on the unit costs presented in table 4. Also presented in table 6 are quantities and costs per foot for full-scale facility interior walls. General characteristics of these intermediate walls are shown in figure 50. These walls would be subjected to vertical loads only, and their proportions are controlled by ACI minimum provisions for load bearing walls. The total costs for the 50 and 100-foot walls include two side walls plus the end wall in the driver section. The cost per foot of test section includes the two side walls only. The cost per foot of interior wall is for a single wall.

Although the propped cantilever retaining wall is probably the least costly, it does have some disadvantages. Since

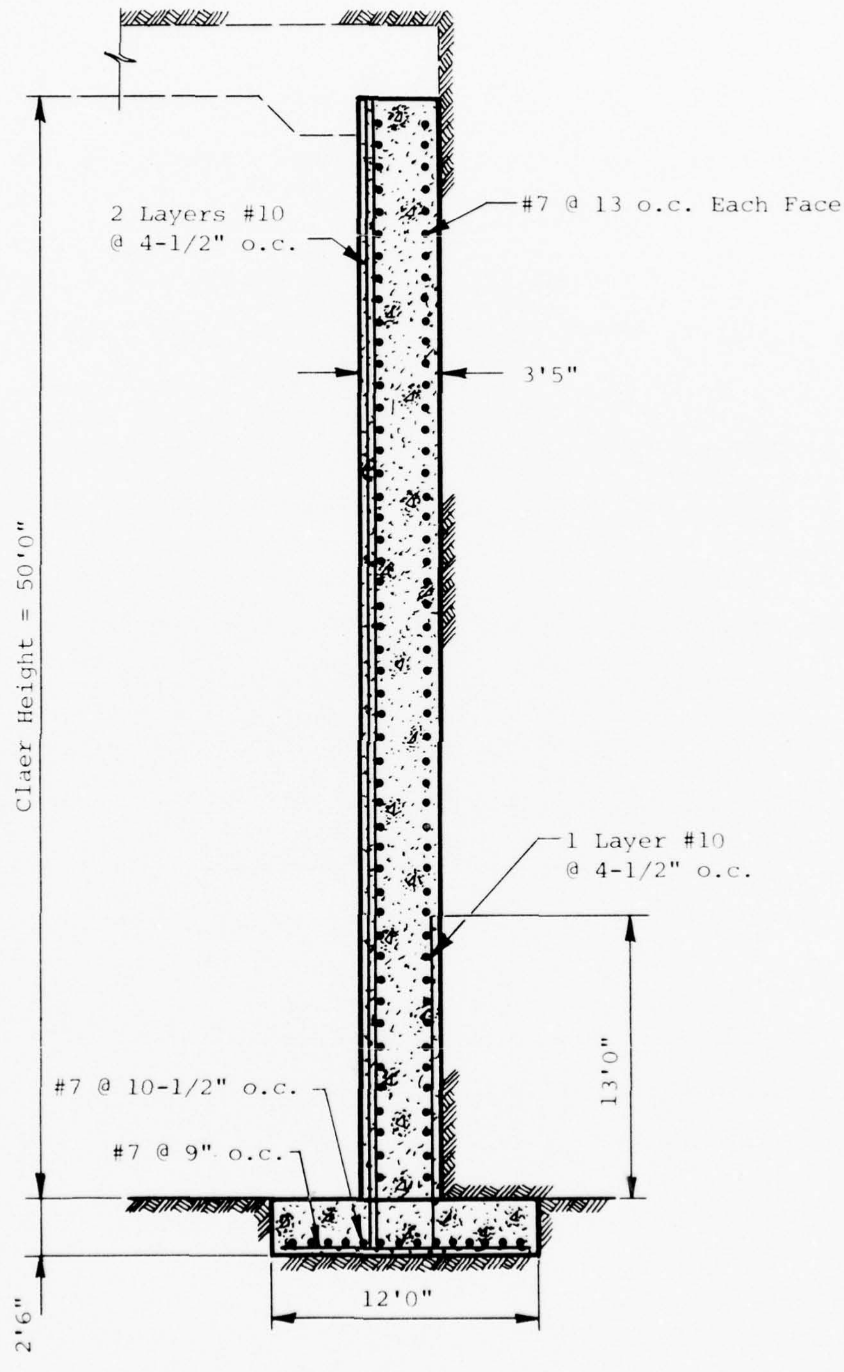

Figure 48. 50-Foot Retaining Wall

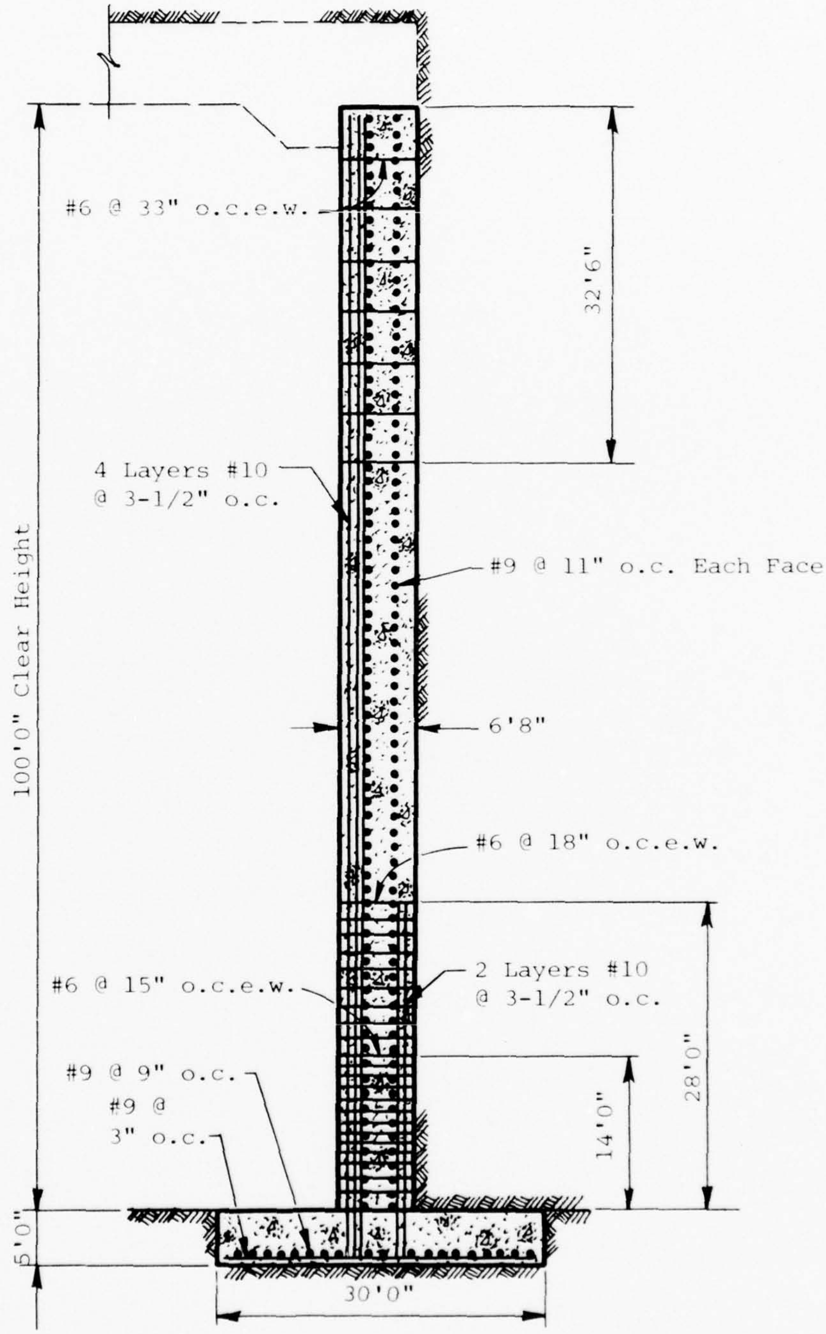

Figure 49. 100-Foot Retaining Wall

Table 6

MATERIAL QUANTITIES AND COSTS FOR RETAINING WALLS

Item	Concrete		Cost Per Foot of Test Section ($000)
	cy	Cost ($000)	
50-Foot Walls for One-Half Scale Facility	6,690	2,007	5
100-Foot Walls For Full-Scale Facility	54,440	16,332	20
100-Foot Interior Walls (1 Wall)	17.9/ft		5

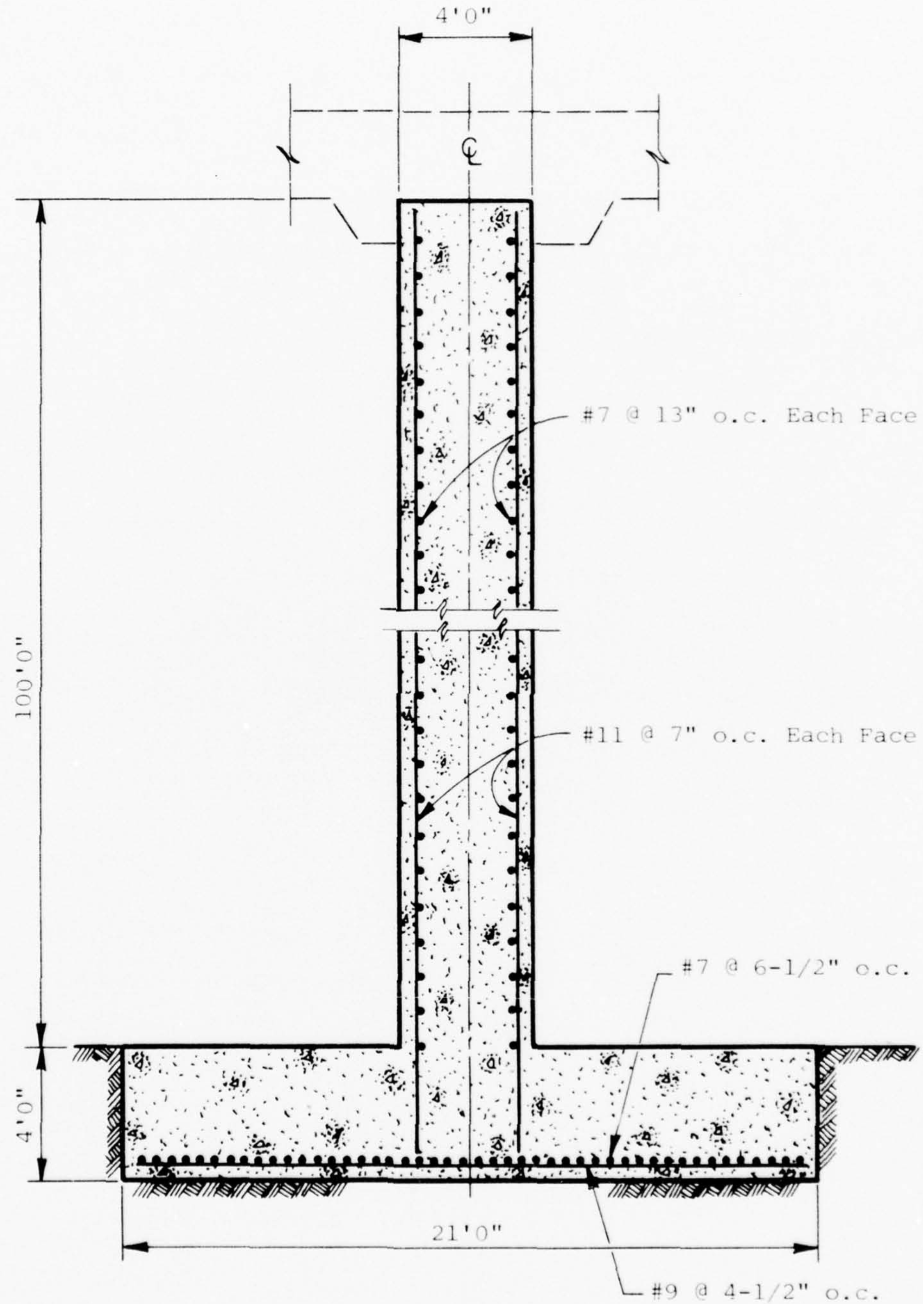

Figure 50. Intermediate Walls for Full-Scale DABS Facility

the wall depends upon the roof structure for stability, backfill cannot be placed behind the wall until the roof is in position. It is also possible that when the roof is blown off, the side walls may rebound and topple onto the test bed. However, previous experience with smaller walls in HEST and DABS events would indicate that this is not a likely occurrence.

4. CIRCULAR CONCRETE ARCH

This concept utilizes a reinforced concrete circular arch to provide the required interior working volume. The depth of the soil cover at the crown of the facility is adjusted to make up the difference between the dead weight of the arch and the required surcharge loading weight. The amount of soil placed on the shelter is kept to a minimum by sloping the fill away from the arch at a 30-degree angle, the predicted angle of internal friction of the soil.

A vertical endwall constructed of reinforced concrete seals the driver end of the arch facility. The mass of the wall is not sufficient to provide the required containment of the large internal pressures generated when the explosives are detonated, so earth backfill is placed against it. Continuous footings support the arch and endwall. Tie beams are included across the base of the arch to restrain the footings horizontally, although a more detailed analysis in a final design may show them to be unnecessary.

Table 7 and figures 51 through 56 summarize the characteristics of one-half and full-scale facilities in the aboveground configurations. The structural details of the arch are the same for both above and belowground construction. All details indicated are preliminary, but the designs are sufficiently complete so that reasonable estimates of material quantities can be obtained.

The one-half scale facility has a uniform thickness around the arch perimeter. Preliminary analysis of the full-scale facility indicates some possible advantage to a change in thickness at 45 degrees from the springline of the arch. Both facilities have increased thicknesses in the driver end due to increased surcharge weight over these sections as shown in figures 51 and 54.

The arch foundations were proportioned on the basis of a sandy subgrade with an angle of internal friction of 30 degrees in lieu of the 4400 pounds per square foot bearing capacity initially specified. Using this approach, the bearing capacity is a function of the footing width, soil friction angle and depth of burial. Although the preliminary designs specify tie beams between foundations, final designs might utilize passive soil pressures to minimize, or possibly eliminate, the requirement for tie beams.

Costs of the various arch options are summarized in table 8. Total costs for the various items are based on unit

Table 7

CIRCULAR CONCRETE ARCH SUMMARY

Cover (in)	Radius (ft)	Minimum Loading (psf)	Crown			Base			Temp Steel
			d (in)	h (in)	Reinforcing	d (in)	h (in)	Reinforcing	
30	50*	500	16	19	#9 @ 6" o.c. p = 0.01	16	19	#9 @ 3" o.c. p = 0.021	#6 @ 8" o.c.
74	50*	1000	24	27	#9 @ 4" o.c. p = 0.01	24	27	#10 @ 3" o.c. p = 0.017	#6 @ 8" o.c.
65	100**	1000	30	34	2 layers #10 @ 4" o.c. p = 0.02	42	47	2 layers #14 @ 5-1/4" o.c. p = 0.02	#11 @ 11" o.c.
155	100**	2000	43	48	#18 @ 4-1/2" o.c. p = 0.02	53	58	2 layers #14 4-1/4" o.c. p = 0.02	#11 @ 11" o.c.

*f_y = 50,000 psi, f'_c = 4,000 psi

**f_y = 60,000 psi, f'_c = 5,000 psi

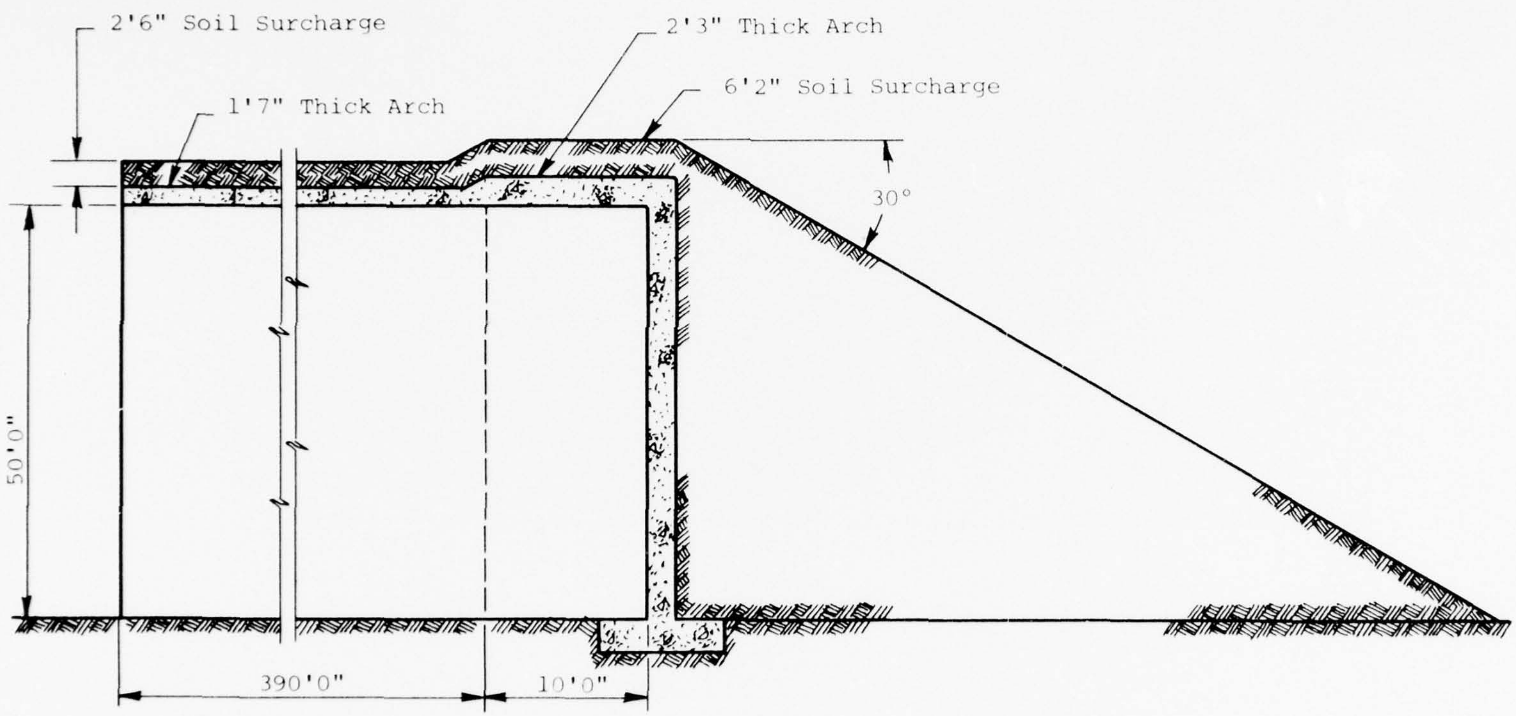

Figure 51. Longitudinal Section Through Aboveground 100-Foot Span Arch

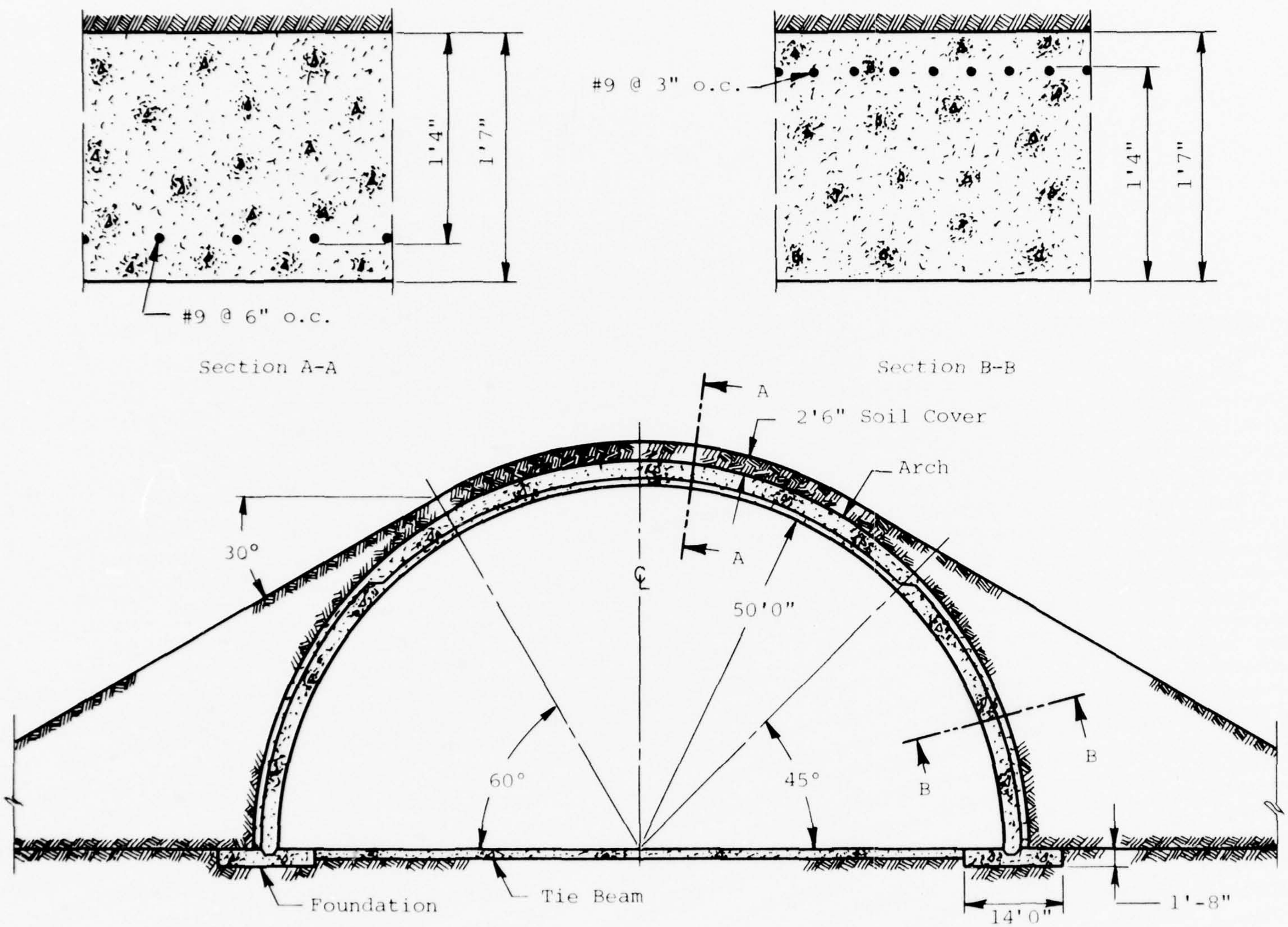

Figure 52. 100-Foot Span Arch for Test Section

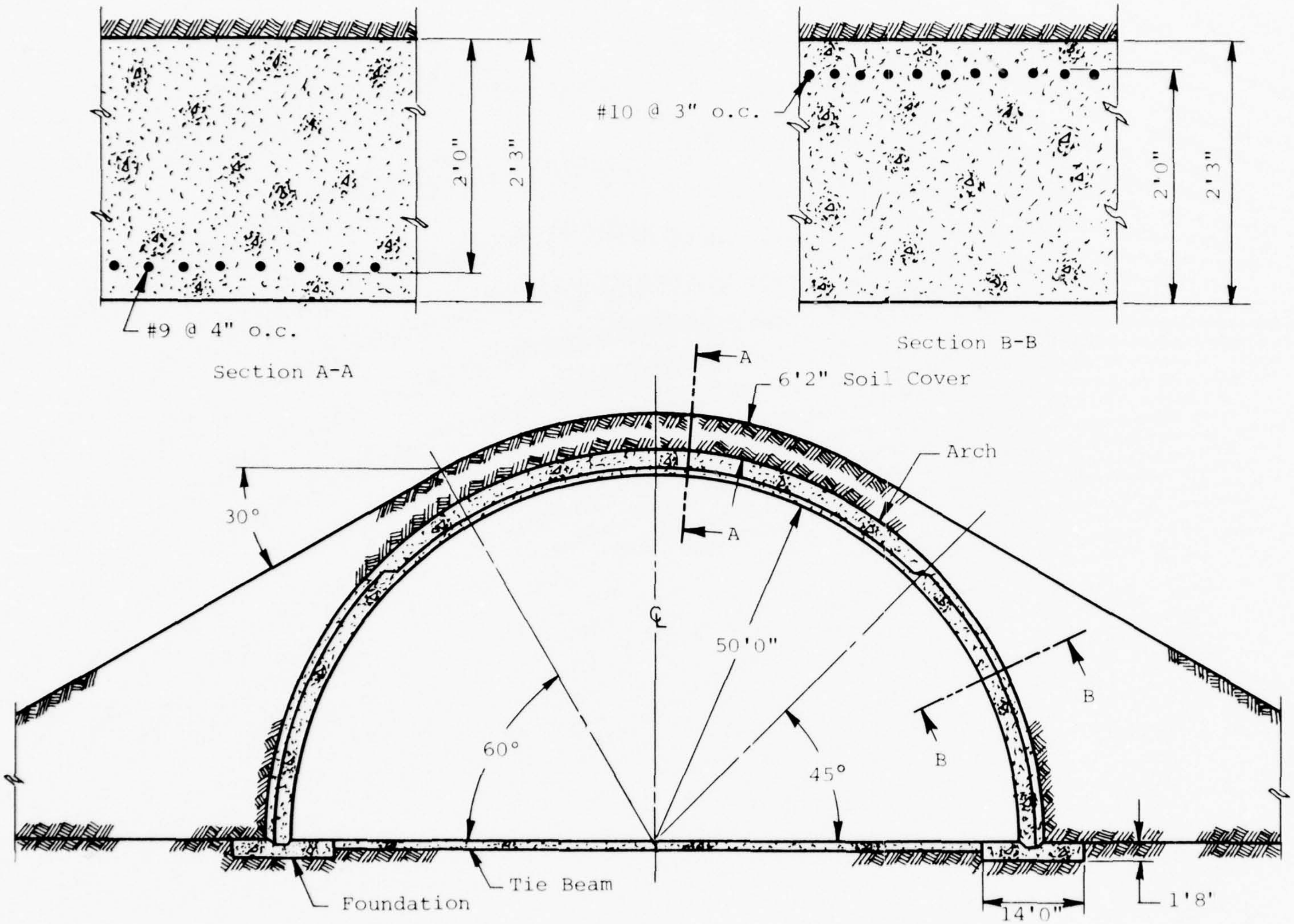

Figure 53. 100-Foot Span Arch for Driver Section

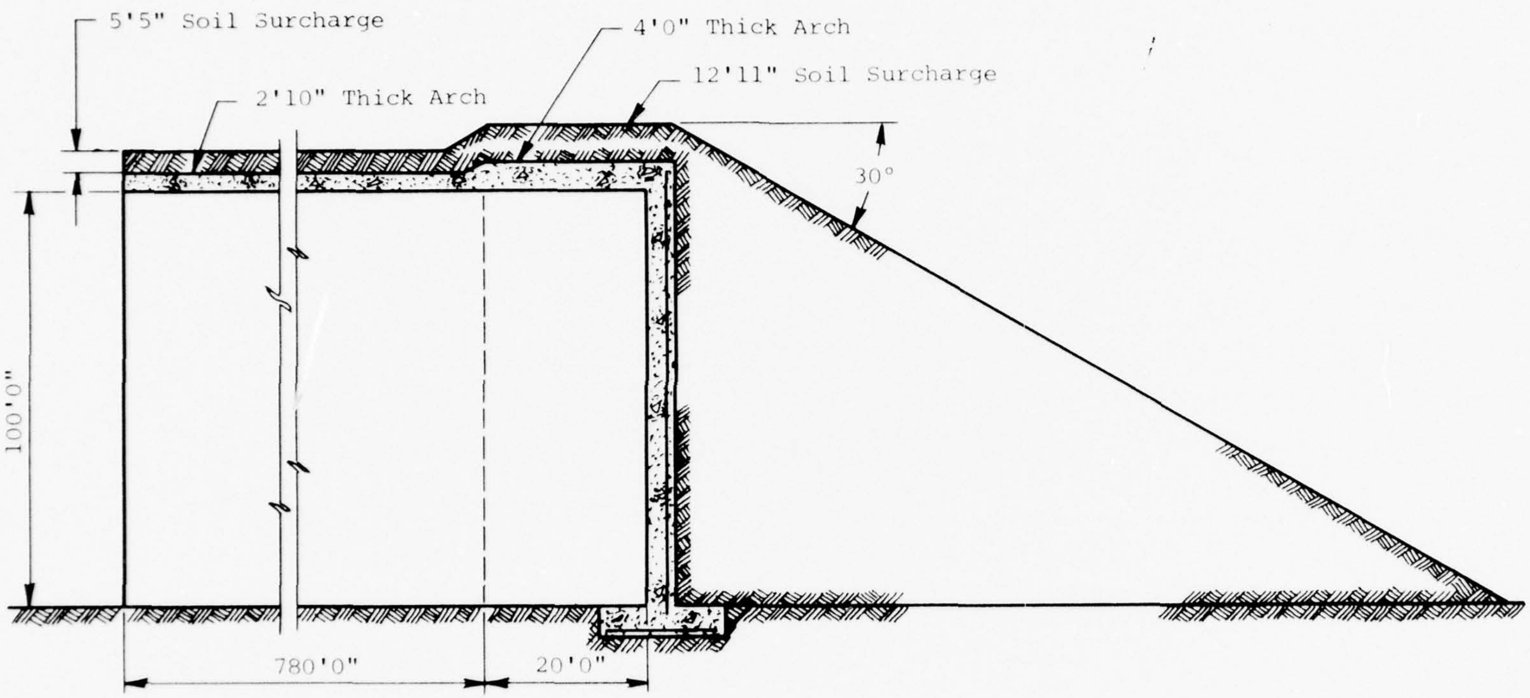

Figure 54. Longitudinal Section Through Aboveground 200-Foot Span Arch

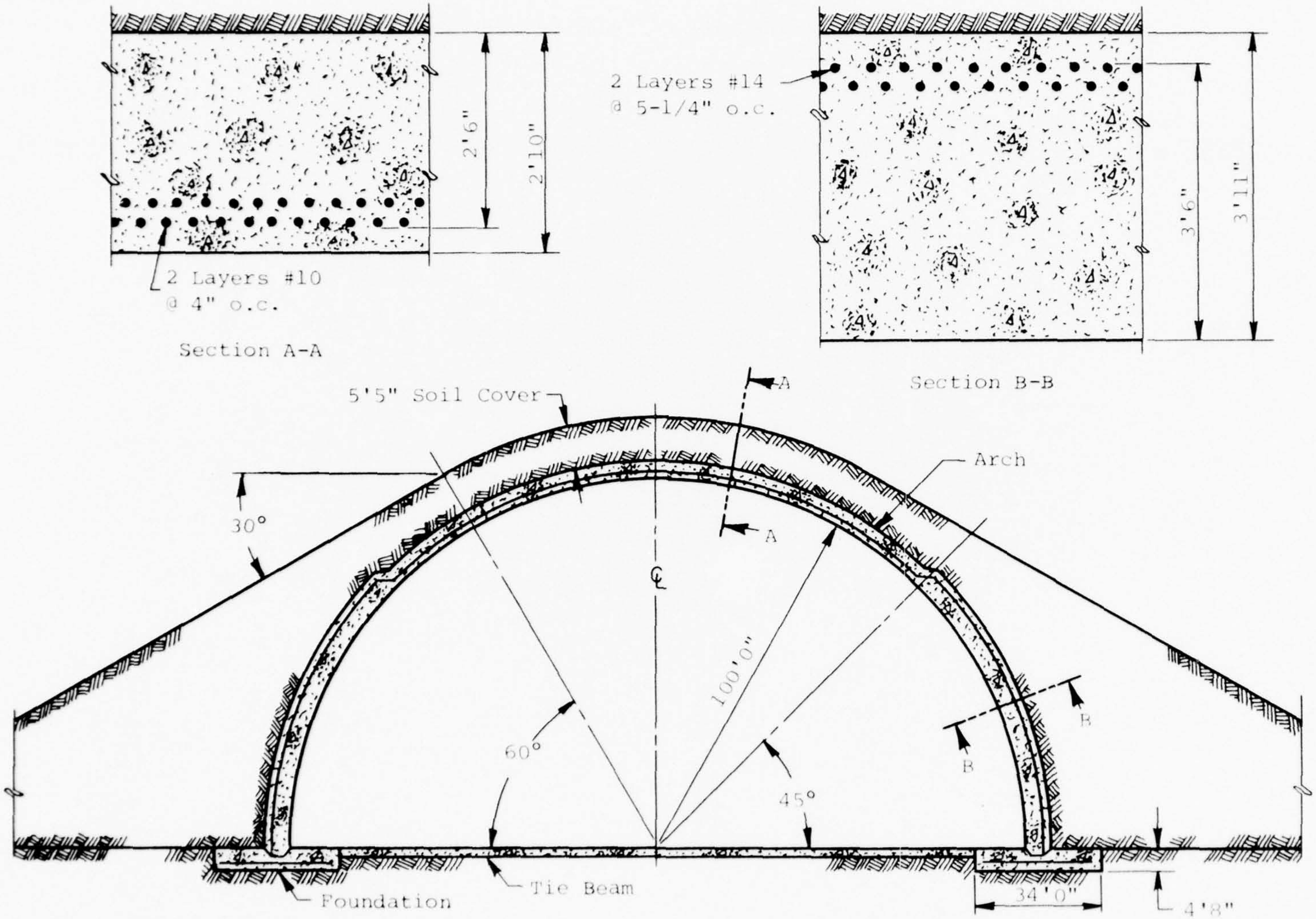

Figure 55. 200-Foot Span Arch for Test Section

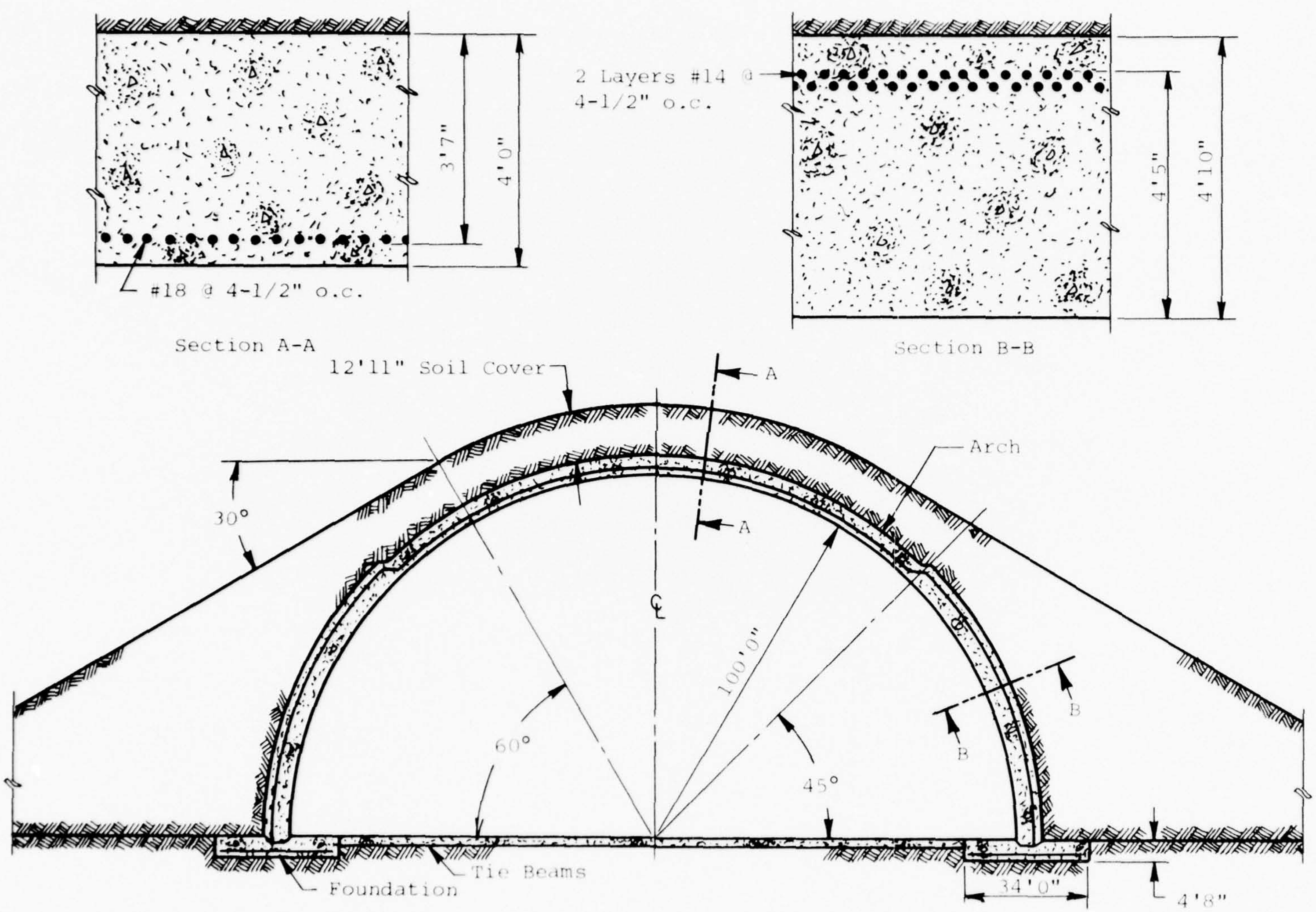

Figure 56. 200-Foot Span Arch for Driver Section

Table 8

MATERIAL QUANTITIES AND COSTS
FOR CIRCULAR CONCRETE ARCH CONCEPTS

Item	Concrete		Structural Steel		Earthwork Cost ($000)	Site Rehab. Cost ($000)	Total Cost ($000)	Cost Per Foot of Test Section ($000)
	cy	Cost ($000)	Tons	Cost ($000)				
A/G 100-Foot Span 400-Foot Facility Length	5,560	1,668	70	105	85	45	1,903	4
B/G 100-Foot Span 400-Foot Facility Length	5,560	1,668	70	105	112	20	1,905	4
A/G 200-Foot Span 800-Foot Facility Length	51,730	15,519	1,067	1,600	702	392	18,213	20
B/G 200-Foot Span 800-Foot Facility Length	51,730	15,519	1,067	1,600	806	147	18,072	20

costs presented in table 4. Earthwork costs were taken from table 5. Quantities and costs include the arch, foundations, endwall, earthwork and site rehabilitation. Site rehabilitation for the aboveground cases assumes that the concrete debris must be hauled to a disposal site. For the cut and fill cases, it is assumed that the debris is buried at the test site. Also presented in table 8 are the costs per linear foot of test section. The minor differences between the above and belowground construction disappear when costs per linear foot are rounded off to the nearest thousand dollars. A change in unit costs would, of course, affect these differences.

The arch concept utilizes structural materials more efficiently than others considered. It minimizes the internal volume of the test facility and, therefore, the quantities of explosives required to produce the desired blast loading. The interior surfaces of the arch are smooth and offer little resistance to flow of the blast wave. A major advantage appears to be lower cost.

The arch concept does have some undesirable features. The size and mass of the arch shell will create construction difficulties. Construction of the arch will require extensive formwork, and this is one of the more difficult costs to estimate. The quantity and cost of the formwork are strongly dependent on the construction plan and schedule. Forming of

thick overhead sections will require substantial internal bracing. One possible alternative would be to construct the arch shell over an earth mound and excavate the interior when the concrete shell has cured. Because of sensitivity to load distribution, care must be exercised in placing heavy surcharge loads on the arches. Overstress of the structure might occur if the soil cover is placed in a haphazard fashion. Unsymmetrical loadings should be minimized during the construction stages.

The arch sections are heavily reinforced, and it is not clear how they will react to the internal blast loading. It is possible that much of the debris could fall on top of the test structures and complicate reentry. It may be possible to promote opening up and sideward motion of arch halves by detonating explosive charges along the centerline at the crown. Another possibility would be the use of a three-hinged arch, as opposed to the two-hinged arch, to promote the opening up and sideward motion of the arch halves. If constructed aboveground, large chunks of reinforced concrete could present a difficult cleanup task, as opposed to construction in the cut and fill configuration where the debris could probably be covered up at the test location.

5. REINFORCED CONCRETE BOX GIRDERS

This concept utilizes reinforced concrete box girders placed on top of vertical retaining walls to provide the

interior working volume required. A soil cover is placed on top of the girders to make up the difference between their dead weight and the specified surcharge weight. A soil backfill with side slopes of 30 degrees is also placed against the side walls (figs. 39 through 42). A soil backfill is placed against the vertical endwall that seals the driver end of the facility. The downstream end of the facility is assumed to be open.

Five different box girder sections have been designed to satisfy various span and surcharge load combinations for the one-half and full-scale facilities. Structural details of the five sections are presented in figures 57 through 61. All designs are preliminary and subject to optimization to obtain more efficient sections. The girder shown in figure 57 would be used over all sections of the one-half scale facility, except the driver section. The one shown in figure 58 would be used over the driver section of the one-half scale facility and could also be used over sections other than the driver section of the full-scale facility, if one intermediate support is provided. Girders of the type shown in figure 59 would provide a clear span of 200 feet over sections other than the driver section. The design shown in figure 60 would be used over the driver section and requires an intermediate support at the middle of the 200-foot span. The girder shown in figure 61 requires two intermediate supports between the

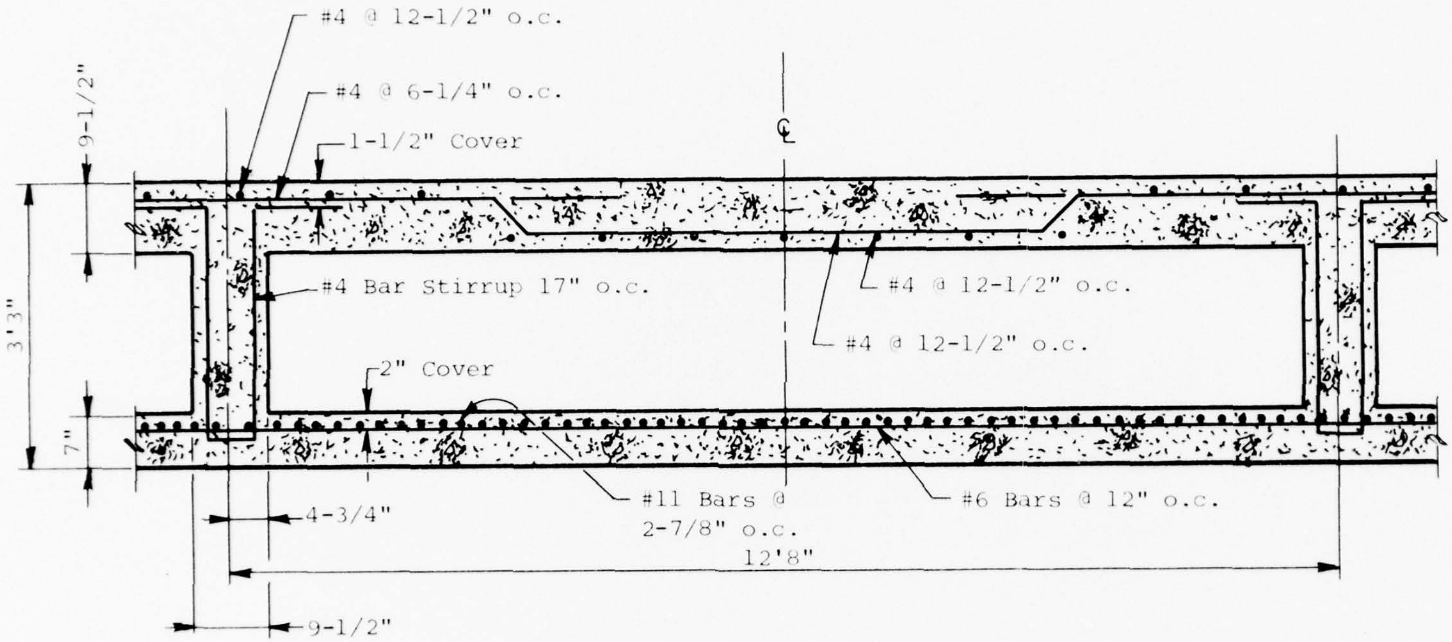

Figure 57. Box Girder for 100-Foot Span 500 psf Surcharge

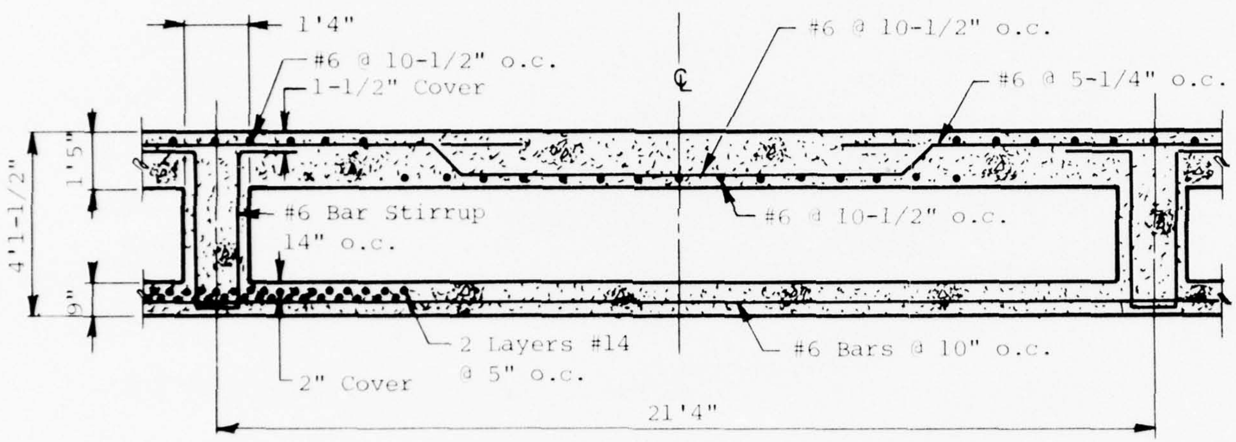

Figure 58. Box Girder for 100-Foot Span 1000 psf Surcharge

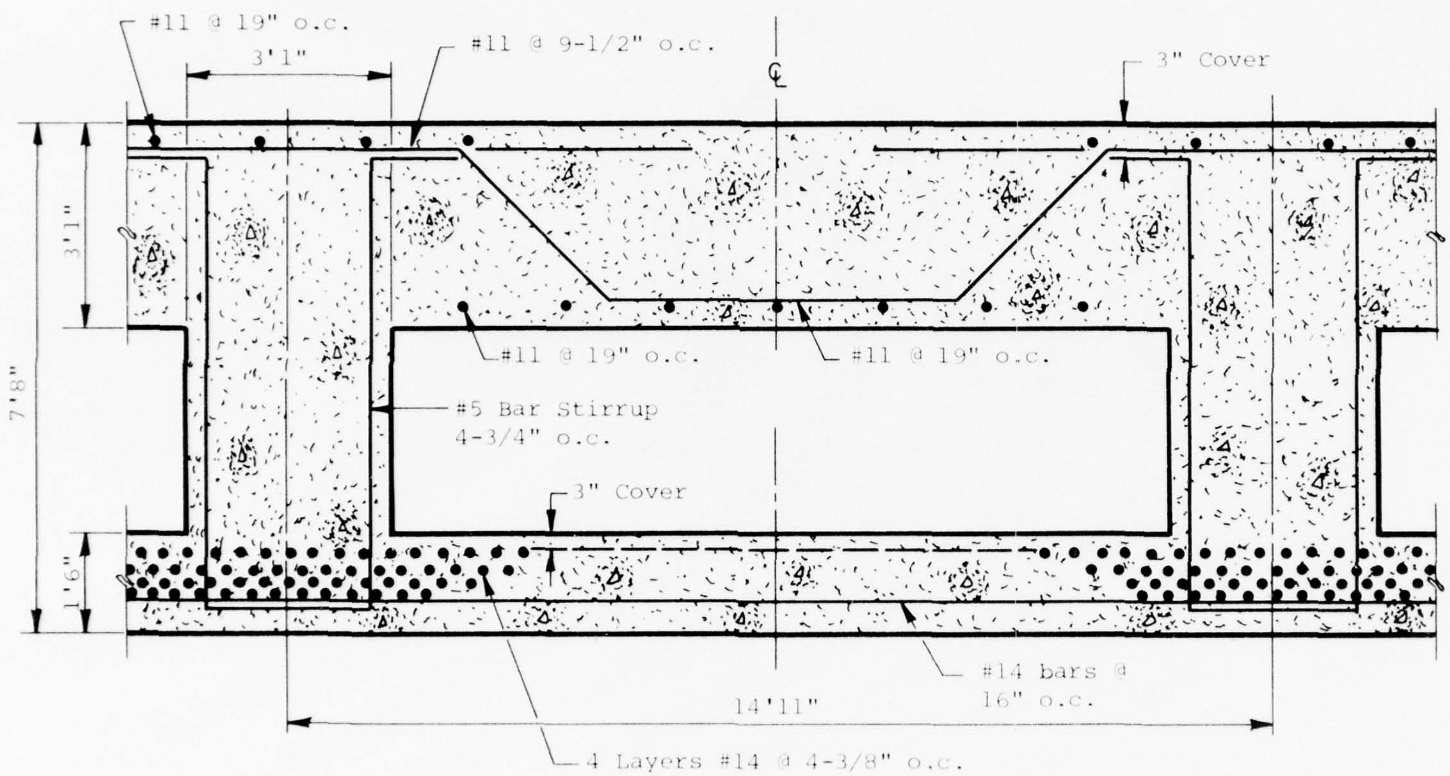

Figure 59. Box Girder for 200-Foot Span 1000 psf Surcharge

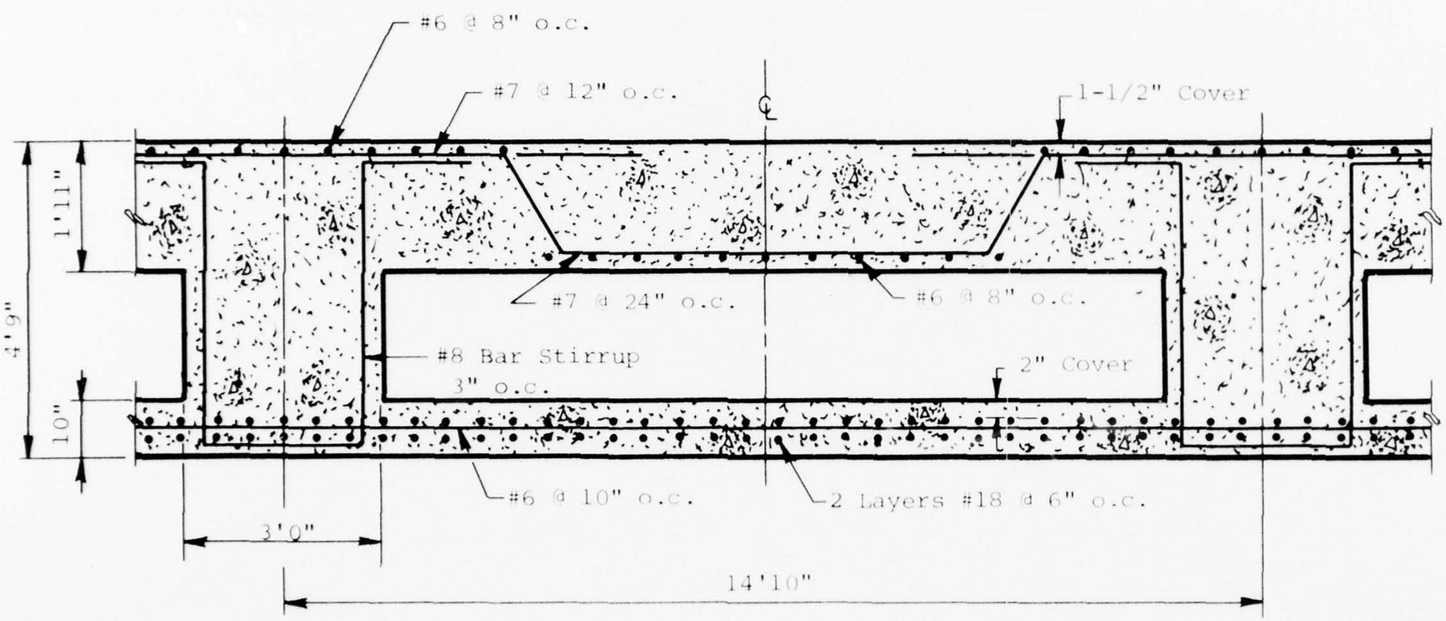

Figure 60. Box Girder for 100-Foot Span 2000 psf Surcharge

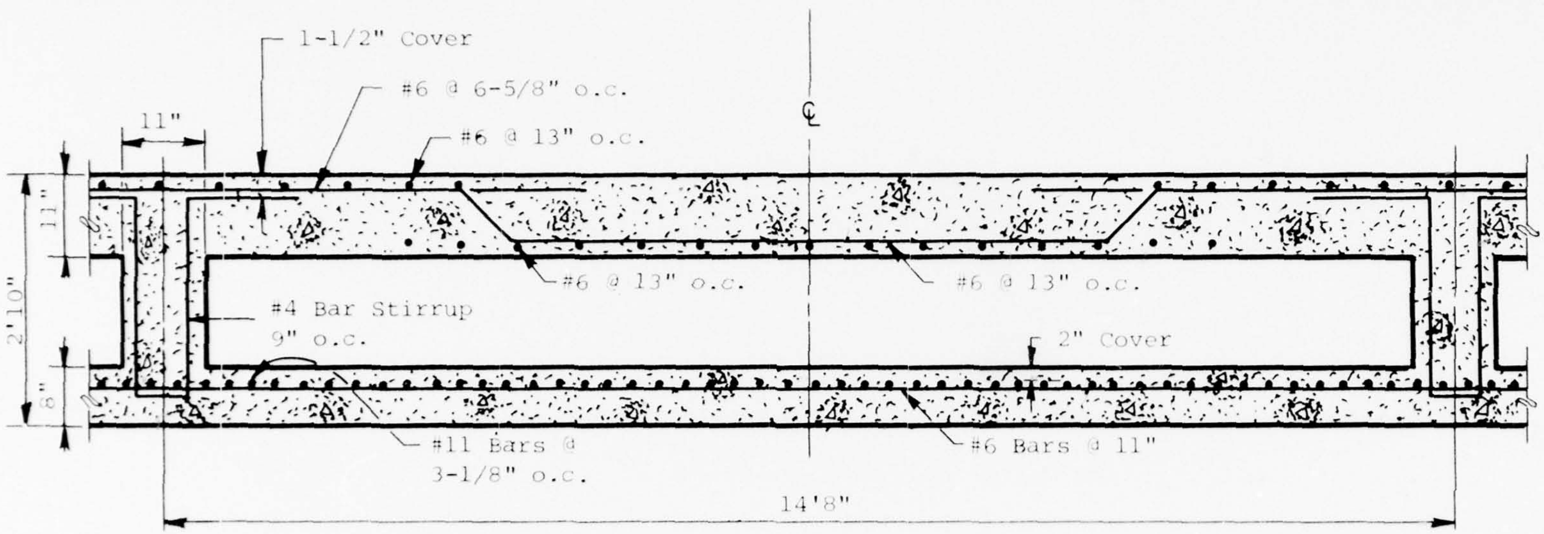

Figure 61. Box Girder for 67-Foot Span 1000 psf Surcharge

outer retaining walls and is used over sections other than the driver section of the full scale facility. Such an arrangement is shown in figure 62. The objective is to reduce roof spans and, at the same time, avoid imposing large concentrated loads on the test item. Although this objective can be achieved for face-on orientation of the MX test shelter, it cannot for side-on orientation of the test shelter. The retaining walls and intermediate supporting walls used in conjunction with this concept are those shown in figures 48 through 50.

Material quantities and costs for various box girder options are summarized in table 9. Earthwork and retaining wall costs are taken from tables 5 and 6, respectively. The options listed in table 9 combine various elements described previously into a complete one-half or full-scale DABS facility. The options are:

- Option 1 - A one-half scale aboveground facility with a clear span of 100 feet. The box girder used over the driver section is that shown in figure 58. The one shown in figure 57 is used over the rest of the facility.
- Option 2 - Same as Option 1 except for belowground construction.
- Option 3 - A full-scale aboveground facility with a clear span of 200 feet over the test section. An intermediate support is provided in the driver section

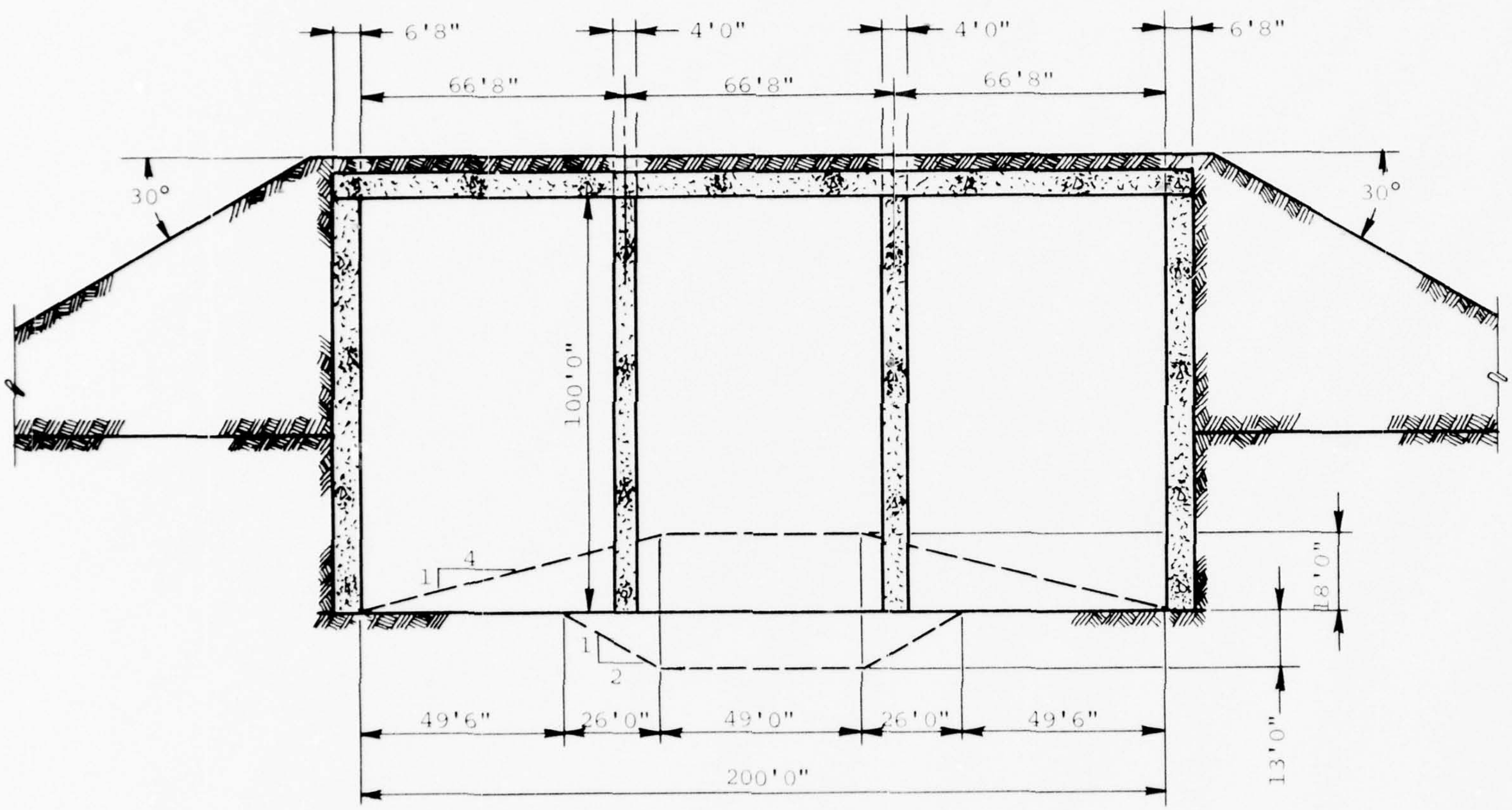

Figure 62. 200-Foot Span with Intermediate Supports

Table 9

MATERIAL QUANTITIES AND COSTS FOR BOX GIRDER OPTIONS

Option	Girder Concrete		Wall Concrete		Earthwork	Site Rehab.	Total	Cost Per Foot of Test Section ($000)
	cy	Cost ($000)	cy	Cost ($000)	Cost ($000)	Cost ($000)	Cost ($000)	
1	2,327	698	6,690	2,007	184	92	2,981	7
2	2,327	698	6,690	2,007	171	31	2,907	7
3	31,856	9,557	54,799	16,440	1,577	821	28,395	32
4	31,856	9,557	54,799	16,440	1,507	274	27,778	32
5	12,866	3,860	72,724	21,817	1,577	817	28,071	30
6	12,866	3,860	72,724	21,817	1,507	274	27,458	30

where the clear span is 100 feet. The girder used over the driver section is that shown in figure 60. The one shown in figure 59 is used over the rest of the facility.

- Option 4 - Same as Option 3 except for belowground construction.
- Option 5 - A full-scale aboveground facility with a clear span of 100 feet in the driver and test sections (other than where the test item is located) and a 67-foot span, as shown in figure 62, over the length of the test item. The girders used are those shown in figures 60, 58 and 61, respectively.
- Option 6 - Same as Option 5 except for belowground construction.

Site rehabilitation costs were estimated by the same method as used for the arch concepts. The costs per linear foot apply to other than the driver section or locations where two intermediate supports are provided. The reduction in cost for the roof system which results from decreasing the clear span in the full-scale facility is largely offset by the cost of the additional walls.

In the initial stages of the study of DABS concepts, reinforced concrete girders appeared to offer some important

advantages over other types of roof systems. Although this is still true in some aspects, a closer look at structural requirements has lessened the attractiveness of reinforced concrete box girders. It was initially assumed that these elements could be precast at ground level and then lifted into position atop the side walls. This approach would greatly reduce construction costs by allowing a large portion of the construction effort to be accomplished at ground level rather than at significant heights above the ground. Preliminary design of a box girder to carry a 1000 psf surcharge over a span of 200 feet has resulted in a system weighing 765 psf. The lightest section, which is designed to carry a 500 psf surcharge over a 100-foot span, weighs 224 psf. As currently designed, precast units for these two systems would weigh about 1200 and 160 tons, respectively. The total weight of these units could be reduced by decreasing their width, but the decreased stiffness could increase the difficulty of handling the units. Shorter spans reduce the weight of the roof system, but only at the expense of providing interior walls. The costs of these walls more than offset any savings in the roof system. Some reduction in weight could be achieved with prestressed concrete, but the net effect on cost is uncertain. As noted earlier, the rectangular cross section of this concept also requires larger quantities of earthwork than the arches.

6. LONG-SPAN STEEL JOIST

This option is similar to the reinforced concrete box girder except that standard long-span steel joists are substituted for the box girders. As noted, the steel joists would not be capable of resisting the loads imposed by a propped cantilever retaining wall and the counterfort wall would be required. Figure 63 shows a section through a one-half scale facility utilizing the long-span steel joists. The 72DLH19 joist is the strongest unit manufactured. For a 500 psf surcharge over a 100-foot span, these joists would have to be placed 17 inches on center. Figure 64 shows one possible arrangement for a full-scale facility. The 1000 psf surcharge over the 67-foot span would require a 15-inch spacing. This spacing would result in a clear distance of only 3 inches between the joist flanges. A 0.75-inch plywood decking would be attached to the trusses to carry the soil surcharge. The roof system would weigh about 60 psf and require a greater depth of soil cover to meet the specified surcharge weight. Reinforced concrete girders or a roof slab appear to be the most practical solutions for a roof over the driver section of the full-scale facility.

Although this option is similar to the box girder concept, it does offer the advantage of using prefabricated units for the roof system. The individual joists are light enough to be easily placed in position on top of the retaining walls.

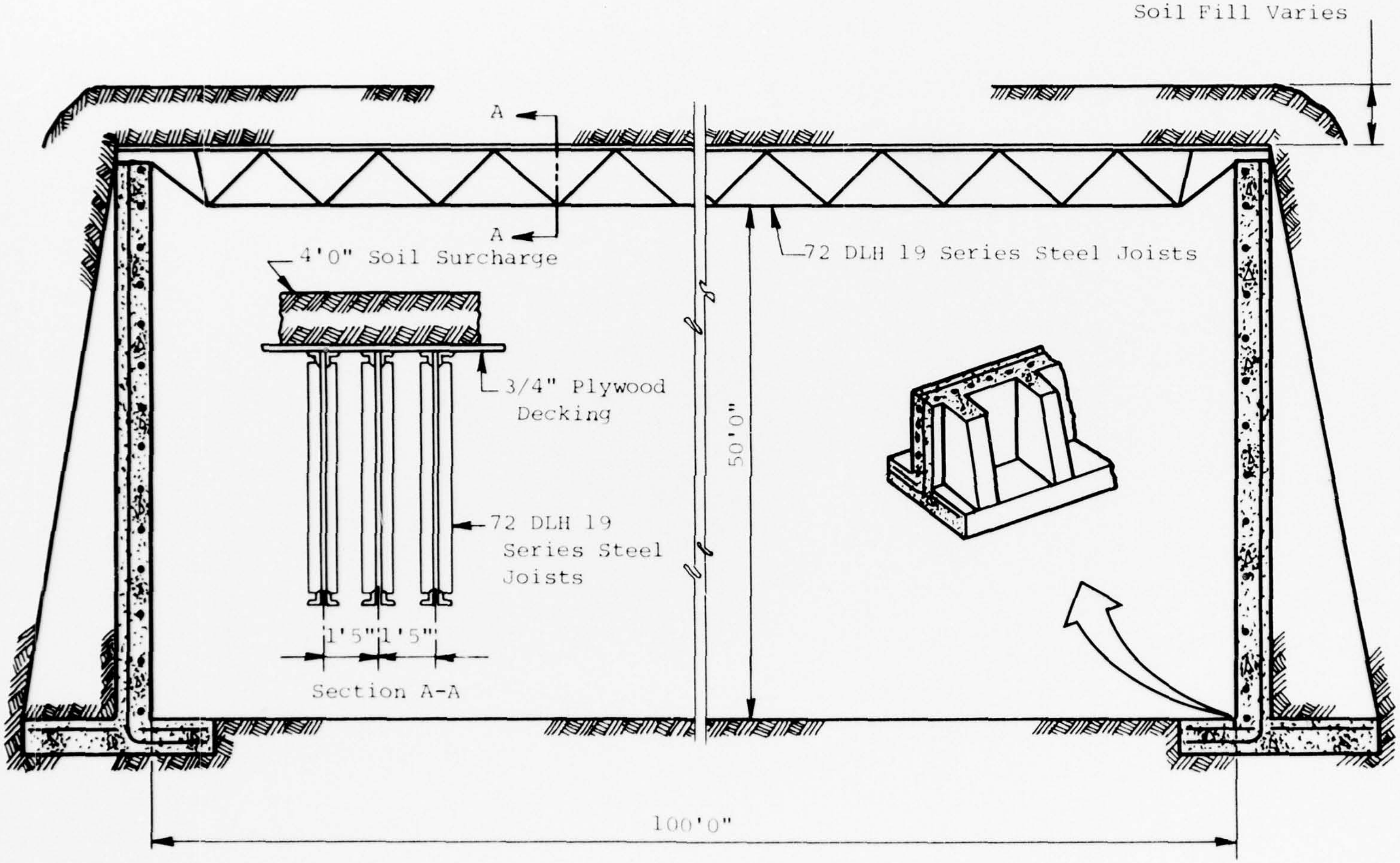

Figure 63. Long-Span Joists - 100-Foot Span 500 psf Surcharge

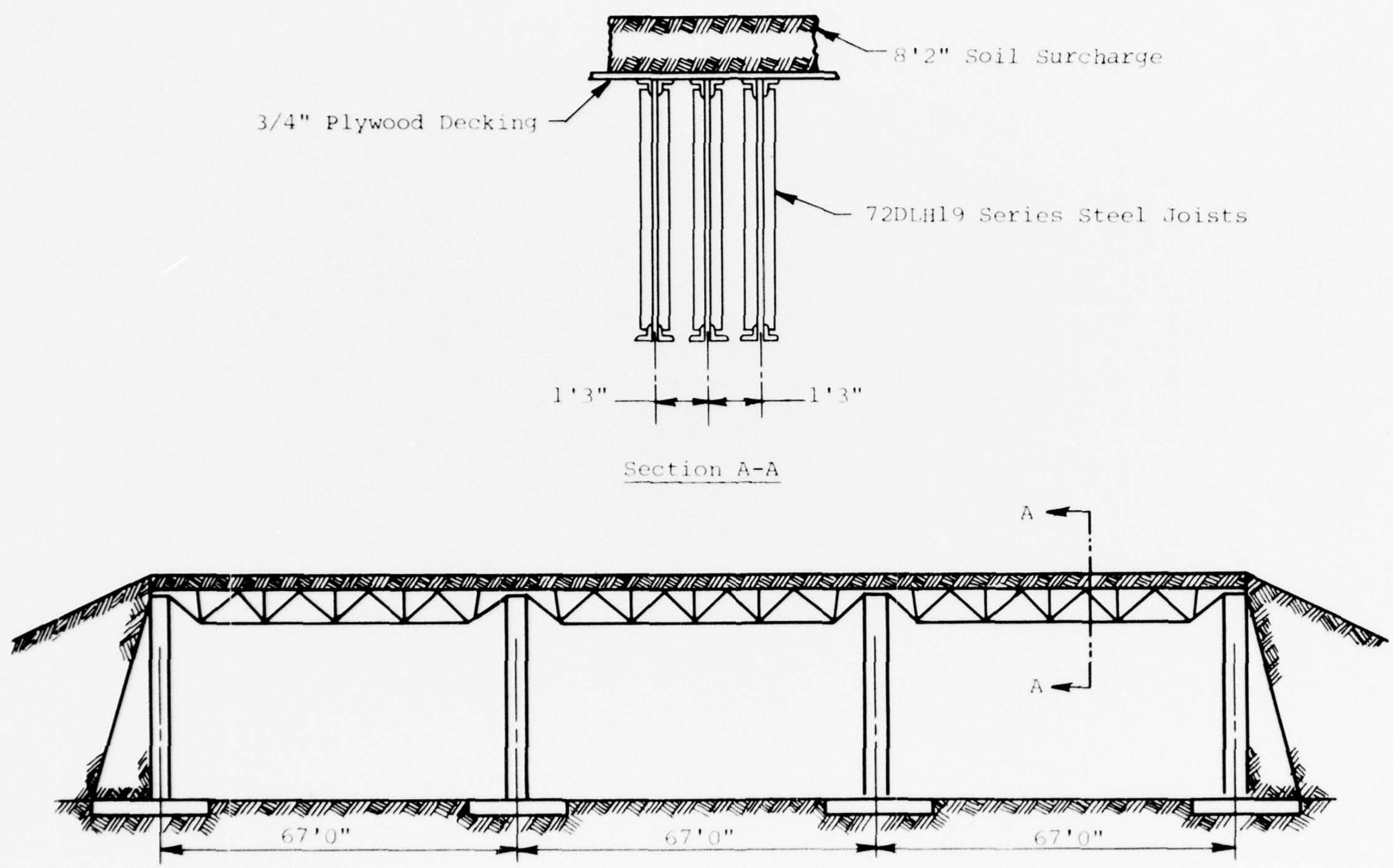

Figure 64. Long-Span Joists - 200-Foot Span 1000 psf Surcharge

While a counterfort retaining wall was not designed for this concept, it would be more costly than the propped cantilever type used for the box girder system because of the greater volume of concrete and the more complicated forming. Since the counterfort retaining wall is a freestanding wall, it could be constructed and backfilled independently of the roof system, unless the roof loads were considered in determining the stability of the wall.

The steel joist option lends itself to debris fallback minimization moreso than the box girders. Whatever debris did fall onto the test bed would be more easily removed than massive chunks of reinforced concrete.

An important undesirable feature of this concept is that the joists project into the interior of the facility. Exposed to the airblast wave traveling through the interior of the facility, they are very likely to be carried downstream and could impact upon test items.

Although a detailed facility design was not completed for this option, the cost of the roof system was compared to that of the reinforced concrete box girders for a one-half scale facility. The total cost for steel joists, plywood decking and additional soil cover was \$2720 per foot of test section. A reinforced concrete box girder for the same span and loading condition would cost approximately \$2550 per foot of test section. When the added cost of the counterfort retaining

wall is considered, the steel joist option does not appear to be cost competitive. No further design effort was spent on the steel joist option because the arch system was even more economical.

SECTION VI

CONCEPT RATING

1. INTRODUCTION

Several options for DABS facilities have been presented. Although they differ in details, they actually represent two basic concepts, a circular reinforced concrete arch and a rectangular structure with a reinforced concrete box girder roof system. Possible evaluation procedures could range from a simple good/poor rating of each pertinent design factor or goal for the various proposed designs to formal application of decision theory. The evaluation procedure proposed is a compromise between the two extremes and corresponds to an informal application of decision theory.

Initially, it was planned to consider rating factors under the two general categories of cost and simulation accuracy. However, the concrete arch and box girder concepts differ only in cross section geometry. Since cross section shape does not appear to significantly affect simulation accuracy, the rating factors pertaining to this category would be equal for both concepts and were dropped from the evaluation procedure. Since both concepts appear to satisfy the technical requirements for a DABS facility, the rating factors are based only on cost. Each factor of merit is assigned a numerical rating between 1 and 10. The design having the greatest overall rating factor (sum of factors of merit) is considered the best design from a cost standpoint.

The following evaluations consider each of the two general DABS structural concepts for both aboveground and balanced cut and fill construction. The HEST facility described in Section III is not a pure DABS facility and was not included in the evaluations.

The rating factors are based on one-half scale facilities and would be generally lower for a full-scale facility because of the higher cost. The rating factors are earthwork, foundation, structural and site restoration.

a. Earthwork

There are relatively small differences in earthwork costs between the aboveground and cut and fill cases for the arches. Both are less than one-half that estimated for the box girders. The arches were given a rating of 10 and the box girders a 7.

b. Foundation

Preliminary foundation designs for arch and box girder concepts indicated smaller costs for the arch. The difference will increase if the wall footing for the girder concept has to be buried deeper, or a key added. The arch was assigned a rating of 10 and the box girder a 9. The aboveground and cut and fill options were considered approximately equal for this factor.

c. Structural

Both arch and box girder concepts will require extensive and substantial internal bracing and formwork. Large

quantities of concrete must be placed with much of the work at significant heights above the ground. The box girder requires about 60 percent more concrete than the arch concept. The arch was assigned a rating of 10 and the box girder a 8. There is little difference between the aboveground and cut and fill cases in this category.

d. Site Restoration

The cut and fill options have an advantage in this category, with the arch having an additional advantage over the box girder because of the smaller quantities of debris. This comparison assumes the concrete debris in the cut and fill cases can be buried at the test site. The aboveground arch was given a rating of 8 and the aboveground box girder a 7. The cut and fill arch was given a rating of 10 and the cut and fill box girder a 9.

2. SUMMARY AND RECOMMENDATIONS

The concept evaluation summarized in table 10 indicates that, on a cost basis, the reinforced concrete arch is superior to reinforced concrete girders supported on vertical retaining walls. The cut and fill arch offers an additional reduction in site restoration costs, if the debris can be buried on site. No significant differences in simulation accuracy are expected between the arch and girder concepts.

Although this study considered only the two-hinged arch, which is continuous from springline to springline, the three-hinged arch with a joint at midspan offers some construction

Table 10

CONCEPT RATING SUMMARY

Concept	Factors of Merit				Sum
	Earthwork	Foundation	Structural	Site Restoration	
Aboveground Reinforced Concrete Arch	10	10	10	8	38
Cut and Fill Reinforced Concrete Arch	10	10	10	10	40
Aboveground Reinforced Concrete Box Girder	7	9	8	7	31
Cut and Fill Reinforced Concrete Box Girder	7	9	8	9	33

advantages. The three-hinged arch is less sensitive to differential settlements; therefore, foundation design criteria are less critical. Precasting one-half sections of the arch is more practical than complete rib sections, and this type of operation could be accomplished at ground level. Precasting arch half-sections could also proceed simultaneously with construction of items in the test bed. This would probably not be possible with cast-in-place arch ribs because of the safety hazards resulting from overhead construction. Precasting operations should be scheduled so as to allow the concrete to develop its design compressive strength prior to erection of the arch ribs. Using this approach, placement of surcharge could begin immediately after erection of the precast sections. Placement of surcharge on cast-in-place sections would have to be delayed until the concrete had developed its required compressive strength.

Precasting operations will have to be closely supervised to insure control of dimensions of arch sections. Such control is necessary if the precast sections are to fit together properly in the assembled arch. The joints between precast sections will have to be grouted or sealed to prevent infiltration of surcharge material. The structural design of the arch sections must consider lifting and erection stresses as well as those resulting from the earth surcharge.

Although not considered in the final evaluation of DABS concepts, the HEST may be a useful technique for some test conditions. Its primary disadvantage is the uncertainty in the degree of simulation it will provide in the case of airblast loading of aboveground structures. The airblast wave generated in a DABS facility is a true shock wave with the associated relationships between peak overpressure and reflected and dynamic pressures. The airblast wave generated in the HEST is not a true shock and exhibits different and largely unknown relationships between overpressure and reflected and drag pressures. In order to use the HEST, the combined pressure loading on the test structure would first have to be established by experimental or theoretical means. A HEST could then be designed to provide this loading distribution, and a limited experimental program conducted to validate the correctness of the design. The HEST is probably best suited for simple test structure geometries where there are no great variations in pressure over short distances.

Assuming the HEST can provide the desired airblast simulation, it can be applied to full scale structures at a much lower cost than the reinforced concrete arch. Although it does apply surcharge loads to the test structure, these loads are insignificant in comparison to the peak airblast pressures and should not affect test results.

The reinforced concrete, three-hinged arch is recommended as the most economical approach to the construction of large

scale DABS facilities. This type of facility has been shown in numerous experiments to provide the desired simulation of nuclear airblast loading on aboveground structures. Subject to verification of its capability of providing the desired airblast simulation, the HEST offers a lower cost alternative for full scale tests where the distribution of airblast loading on the structure has been predetermined by other means.

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